SEWAGE AND INDUSTRIAL WASTES

(Formerly Sewage Works Journal)

JOURNAL of the FEDERATION OF SEWAGE AND INDUSTRIAL WASTES ASSOCIATIONS

Special Features

B.O.D. and Total Load—Buswell

Treatment of Anodizing Wastes—Corcoran

Phosphorus and Algae—Curry and Wilson

Trickling Filter Fungi—Feldman

Activated Sludge Operation—Torpey and Chasick

Responsibility of Designer—Brown and Caldwell

NOVEMBER, 1955—VOL. 27, No. 11

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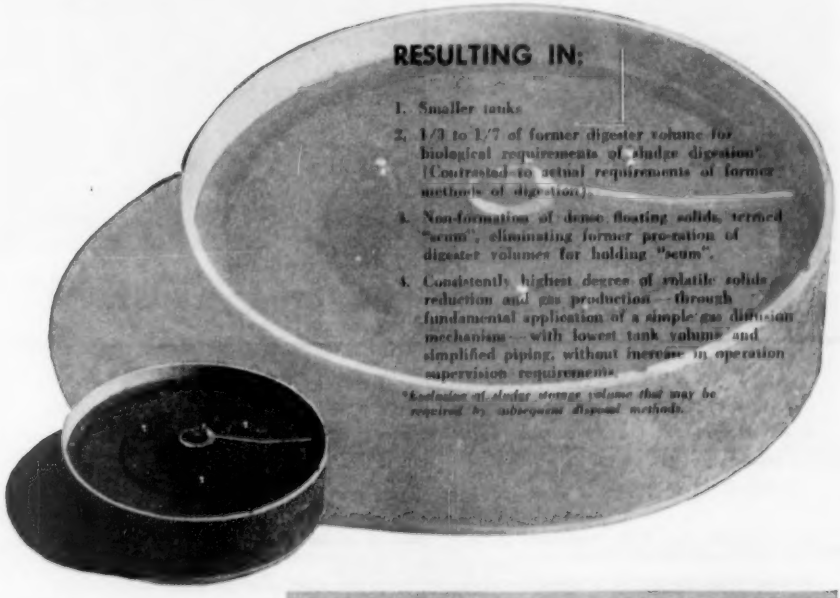
CATALYTIC
REDUCTION
PROCESS

provides exclusively highest solids loading for sludge digestion


RESULTING IN:
1. Smaller tanks
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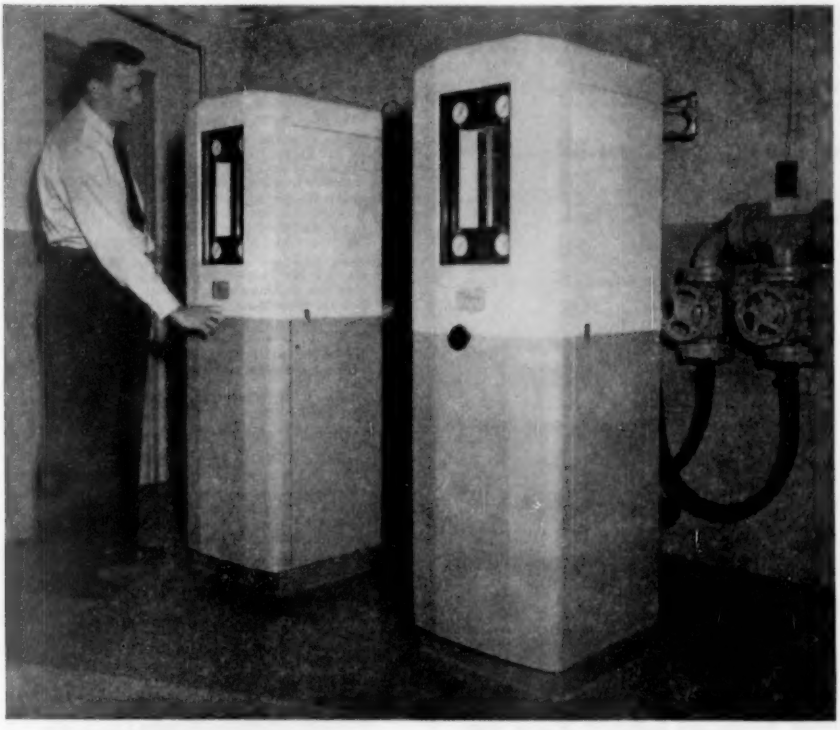

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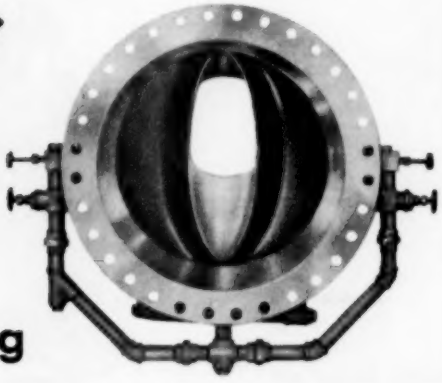

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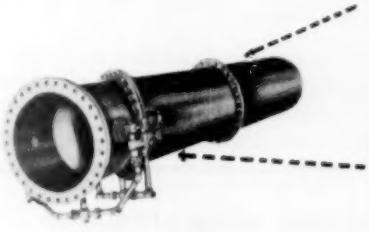

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Association	*Place*	*Time*
Nebraska Sewage and Industrial Wastes Assn.	Yancey Hotel Grand Island, Nebr.	Nov. 3–4, 1955
Florida Sewage and Industrial Wastes Assn.	Orange Court Hotel Orlando, Fla.	Nov. 6–9, 1955
North Carolina Sewage and Industrial Wastes Assn.	Robert E. Lee Hotel Winston Salem, N. C.	Nov. 14–16, 1955
New York Sewage and Industrial Wastes Assn.	Belmont-Plaza New York City, N. Y.	Jan. 18–20, 1956
New Jersey Sewage and Industrial Wastes Assn.	Hotel Traymore Atlantic City, N. J.	Mar. 14–16, 1956

TWENTY-NINTH ANNUAL MEETING

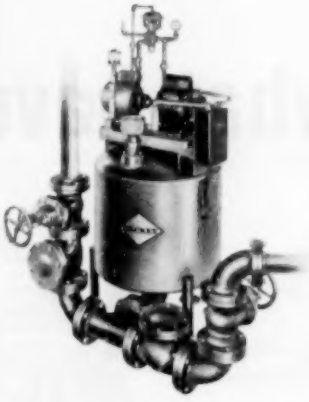
Federation of Sewage and Industrial Wastes Associations

in conjunction with

California Sewage and Industrial Wastes Association

Statler Hotel, Los Angeles, California

October 8–11, 1956



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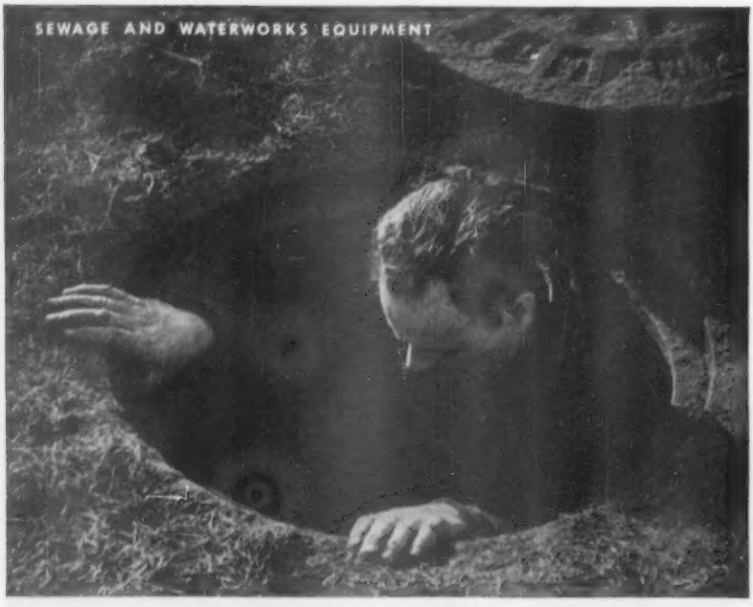

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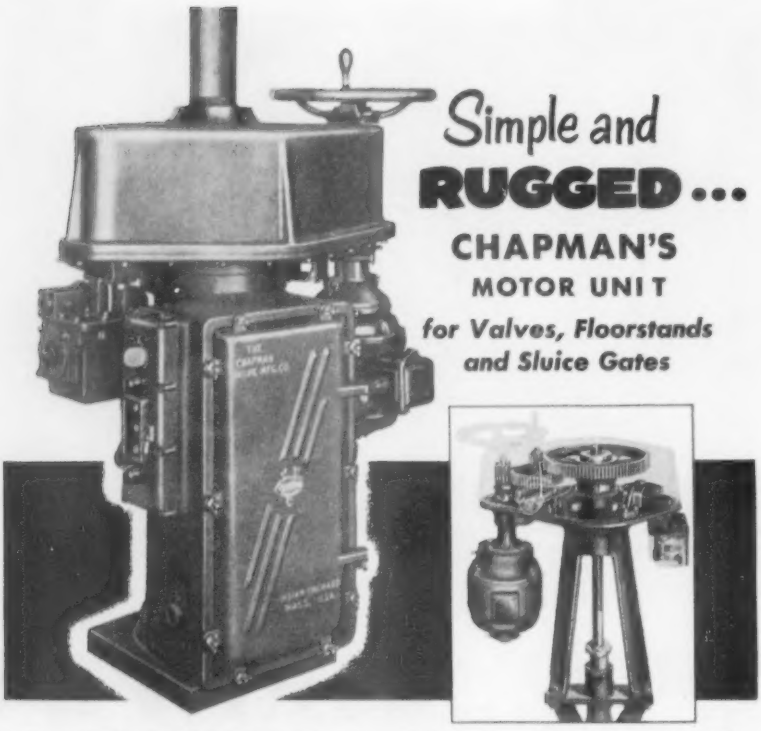

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Springfield Sewage Disposal Plant

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for some time past, we recently changed our operation and
began burning a good use for surplus gas. Thanks to the
flexibility of our equipment we are able to take advantage
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very truly yours,
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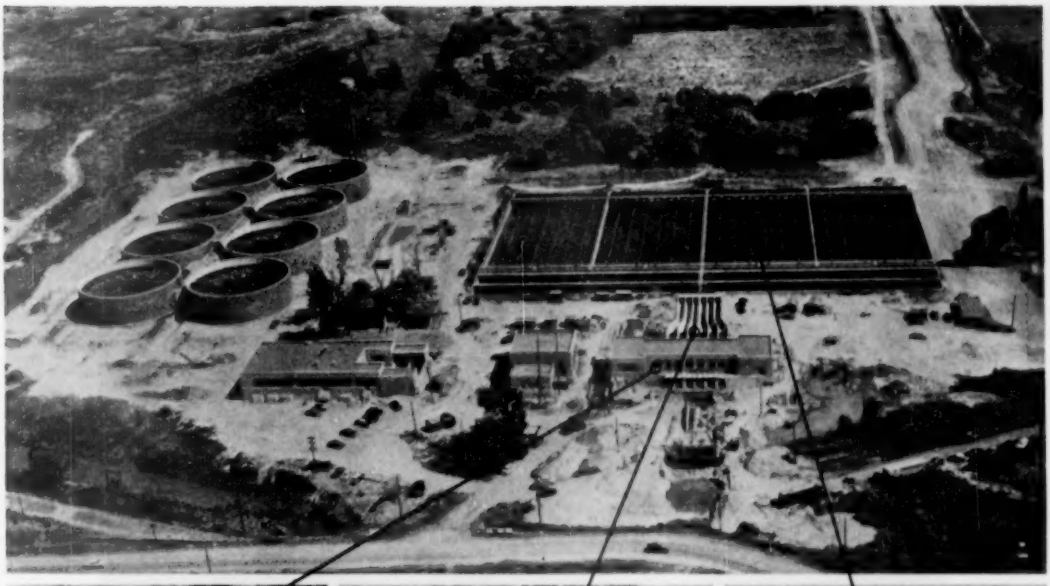

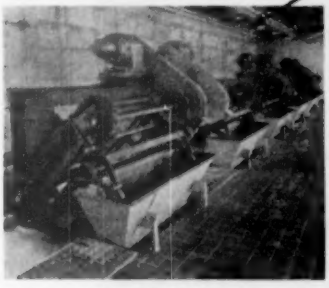

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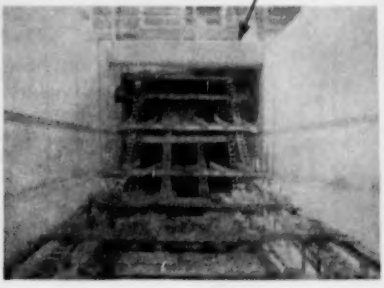

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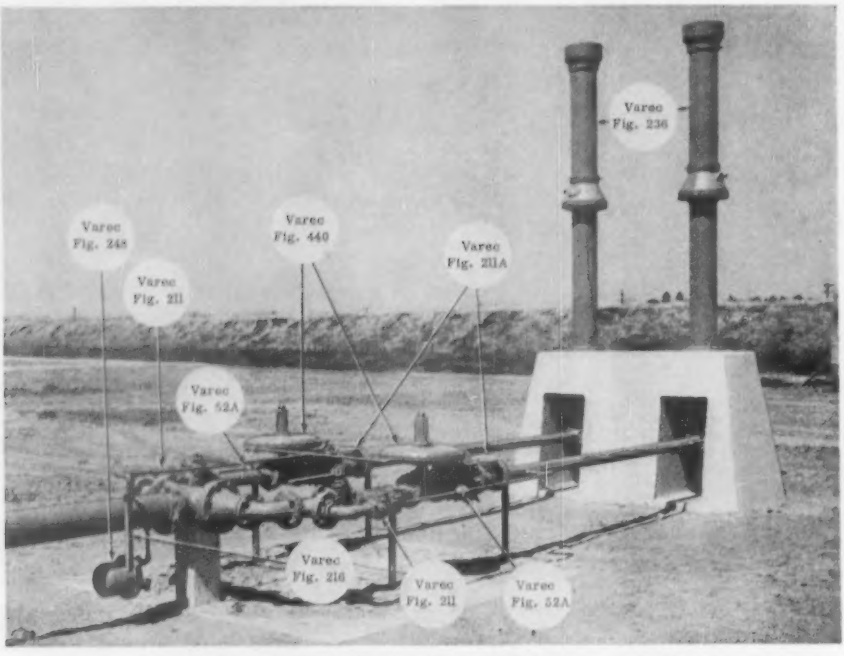

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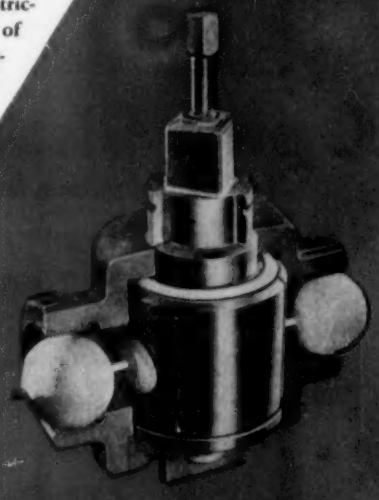

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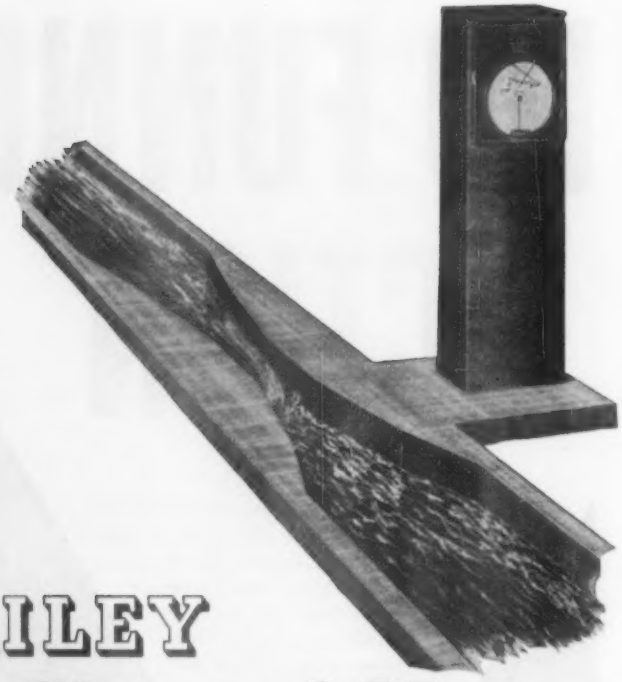

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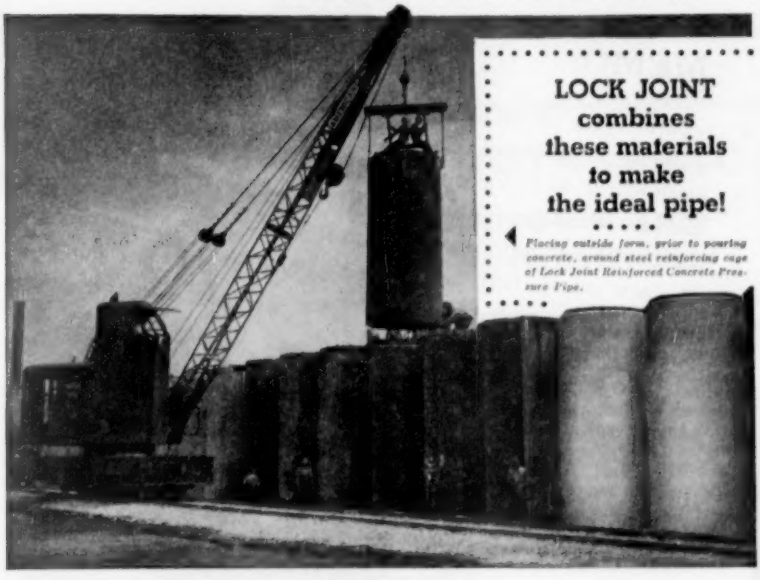

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Sanitary Engineers' Notebook

MEMO:

Several types of vacuum pumps may be used in sludge dewatering installations: Centrifugal water-sealed types such as Nash Hytor: Rotary positive types with or without water seal, such as Roots-Connersville: Piston type dry vacuum pumps, such as Worthington or Ingersoll-Rand. Specific conditions of each installation dictate the choice of vacuum pump. Each has its proper place, depending upon the size and layout of the installation.

A mixture of filtrate and air is drawn through the filter by the vacuum pump. For COILFILTERS with an area of 85 or 100 square feet, a special vacuum pump is used which handles both air and filtrate. For larger installations, the two are usually separated in a vacuum receiver installed between the filter and the vacuum pump. Air is exhausted by the vacuum pump from the top of the receiver and filtrate is pumped out of the bottom. A self-priming centrifugal pump with water-sealed stuffing box and a ball check valve on the discharge is most dependable for filtrate service.

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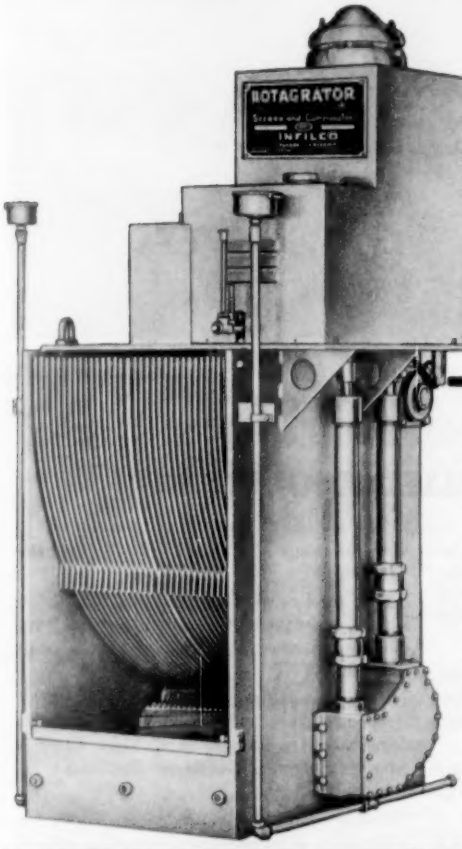

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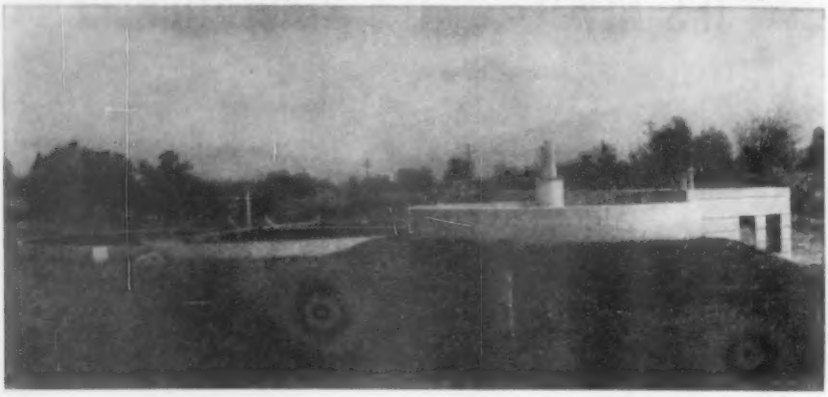
Left, PFT Rotary Distributor for 85' filter bed; right, 35' digester with PFT Floating Cover



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A modern trickling filter plant adopts

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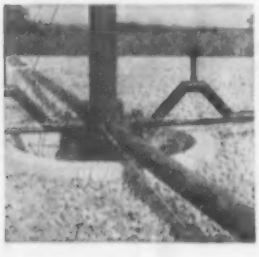
Early in the planning stage, PFT assisted Pittsfield's consulting engineers by showing the adaptability of its equipment in meeting specific plant requirements.



PFT Rotary Distributor showing rugged construction of center column and Low-Loss Overflows on secondary arms.

Design of plant by / *Warren & Van Praag, Consulting Engineers, Decatur, Illinois*

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SEWAGE AND INDUSTRIAL WASTES

Journal of the Federation of Sewage and Industrial Wastes Associations

Volume 27 **November, 1955** **Number 11**

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Ralph E. Fuhrman, *Editor*, 4435 Wisconsin Ave., N.W., Washington 16, D. C.
Dr. F. W. Mohlman, *Advisory Editor* Donald P. Schiesswohl, *Assistant Editor*
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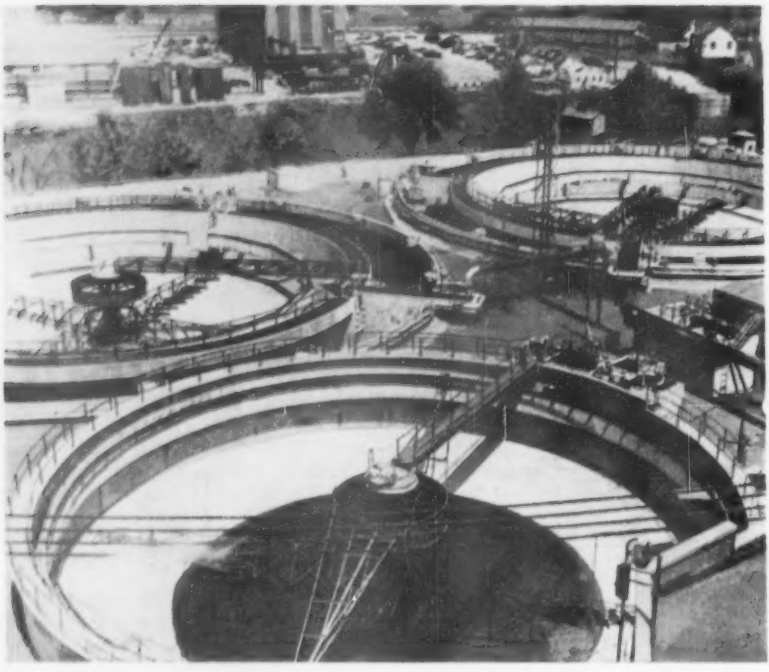

General view of the plant during final stages of construction. Dorr Primary Thickener is in foreground with Currie Clarastors in rear.

Consulting Engineers: GIBBS & HILL, INC., New York, New York

West Virginia Pulp and Paper Company Pioneers New Waste Treatment Process

Dorr-Oliver Equipped Plant Sets Pattern for Industry

Covington, Va. Industrial waste treatment has taken a major step forward with the start-up of this new plant at West Virginia Pulp and Paper Company's Covington, Va., mill. A "first" for the alkaline pulping industry, it points the way toward a solution of these tough-to-handle wastes. Two years of pilot plant work went into the final design and the Covington mill's River Research Group worked closely with the National Council for Stream Improvement throughout all phases of the project. The firm of Weston, Eckenfelder and Hood was also called in to make an intensive round-the-clock analysis of pilot plant results.

The flowsheet as developed is unique. Basically it employs the activated sludge process, one of the standard methods for treating municipal sewage. A departure from standard practice is the addition of certain nutrients essential to the life of the bacteria which carry out the treatment process. Another is split treatment providing primary sedimentation for the portion of mill wastes containing solids and then parallel aeration and final sedimentation for the entire flow. A third is the use of complete instrumentation with one control panel governing the process and recording results.

Average treatment plant design is 16 MGD with a maximum capacity of 25 MGD. Major equipment units include a Dorr Primary Thickener and two Currie Clarastors.* These latter units employ Dorr Final Clarifiers surrounded by annular aeration channels. In this way two distinct phases are carried out in a single unit. When the plant reaches full operation an estimated BOD reduction of 80-90% will be made in the mill effluent being discharged to the Jackson River.

Dorr-Oliver Incorporated, Stamford, Conn.

*Trademark, Reg. U. S. Pat. Off.

Sewage Works

PRINCIPLES OF ACTIVATED SLUDGE OPERATION

By Wilbur N. Torpey and A. H. Chasick

Respectively, Engineer of Treatment and Associate, Bureau of Sewage Disposal Operation, Department of Public Works, New York City, N. Y.

Considerable attention has been devoted to the theoretical aspects of the activated sludge process and the design of the required units. On the other hand, less material has been published regarding principles relating to the control of the process and its modifications. For a number of years the authors have been directly concerned with the details of treatment methods for obtaining effective and stable operation with due consideration for economy. This comprised control over various modifications of the activated sludge process at eight biological treatment plants treating from 11 m.g.d. to more than 200 m.g.d. of domestic sewage. From observation and comparison of the performance of these plants certain concepts and procedures were derived which have been useful in the control of the activated sludge process. It is believed that these concepts may be of assistance in establishing treatment criteria suitable to conditions found in other localities.

Conventional Activated Sludge

During the course of many years of operation, the activated sludge plant has been developed and standardized. The principal features of the conventional activated sludge sewage treatment plant are shown in Figure 1. For essentially domestic sewage from communities with normal per capita water consumption rates, the requirements are about as follows: preliminary sedimentation for about one hour; aeration for six hours, based on sewage flow plus 25 per cent return sludge; and final sedimentation at an overflow rate of 1,000 gal. per sq. ft. per day. A plant so designed may be expected to remove about 90 per cent of the suspended solids and B.O.D. and to perform with reasonable stability with proper operation.

The operating controls of a conventional activated sludge plant are comparatively limited. These include the air supply rate, return sludge rate and aerator solids concentration. Experience has shown that the air should be adjusted to maintain 2.5 to 5 p.p.m. of dissolved oxygen in the liquor at the aerator outlet; that return sludge rates in the order of 25 to 35 per cent of the sewage flow are satisfactory; and there will be a restricted range of aerator solids concentration which will produce high quality effluents and promote process stability.

Reducing Aerator Volume

In conventional aeration systems it often has been demonstrated that the clarification of the sewage is accomplished in the first small fraction of the aeration tank. Why, then, are such large volumes of aeration tank required? What would be the effect of attempting to operate with one-half the usual volume of aeration tank?

Consider an activated sludge plant treating normal domestic sewage, as

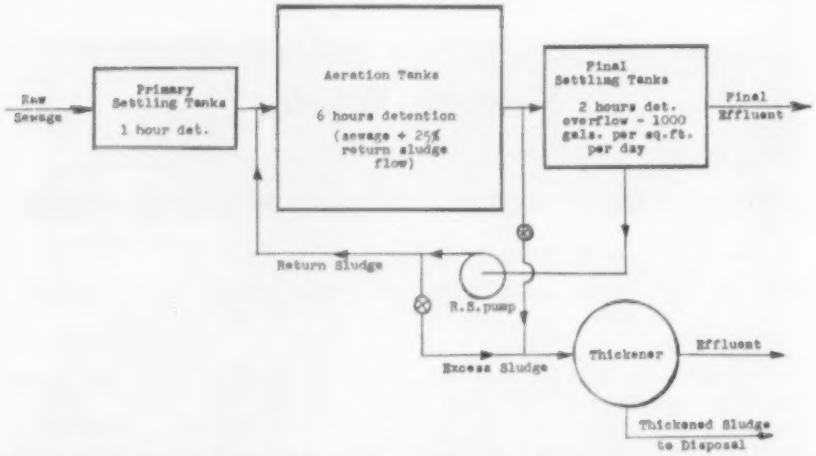

FIGURE 1.—Flow diagram of a conventional activated sludge sewage treatment plant.

illustrated in Figure 1, with an aerator liquor concentration of 1,000 p.p.m. of suspended solids. Suppose one-half of the aeration tanks were taken out of service, keeping the other operating factors, such as air ratio, return sludge rate and aerator solids concentration the same, while treating the same sewage flow. Within about a week the activated sludge will become so voluminous and its settling properties so impaired that the process fails.

Sludge Age

A straightforward explanation of the above phenomenon is that the reduction to one-half of the previous aerator volume also cuts the weight of activated sludge by one-half. This reduced quantity of culture was unable to accommodate the load and still retain its normal biological and physical properties. One measure of this ratio of culture to load was formulated by R. H. Gould, who designated it as "sludge age." This term may be defined as the average time a particle of suspended solids remains under aeration. It is expressed mathematically as the dry weight (pounds) of activated sludge in the aeration tanks divided by the suspended solids load (pounds per day) in the sewage entering the aeration system. Based on a primary effluent concentration of 90 p.p.m. of suspended solids, the activated sludge system with a 6-hr. aeration tank was therefore operating at 3.5 days sludge age. By reducing aerator volume by one-half and maintaining the same suspended solids concentration, the sludge age was lowered to 1.75 days, which was below the critical limit for stable operation. Can the 3-hr. aerator be operated and still maintain adequate sludge age?

Consider the plant with 6-hr. aeration tanks, having an aerator solids concentration of 1,000 p.p.m. and operating satisfactorily at a sludge age of 3.5 days. Suppose one-half of the aeration tanks are taken out of service and all the activated sludge solids transferred to the other half. The concentration there will increase to 2,000 p.p.m. and can be maintained at that level, thus keeping the sludge age unchanged. This heavily loaded system will function only as long as the settling properties of the activated sludge remain excellent. It should be pointed out that the same final settling tanks are now receiving activated sludge solids at twice the concentration and, hence, twice the former rate. If attempts are made to compensate

for this higher solids loading by increasing the return sludge rate, it can be demonstrated mathematically (see Appendix) that to maintain the previous return sludge solids concentration would require an increase in the return sludge flow-rate from the former 25 per cent to 72 per cent. If the 6-hr. aerator system normally required 35 per cent return sludge flow, in order to accommodate double the rate of solids addition to the final tanks the return flow would have to be increased to 119 per cent. The average activated sludge plant does not have the facilities to meet these conditions. The usual outcome would be the accumulation of a deep sludge blanket of activated solids in the final tanks. Prolonged detention there in the absence of oxygen would cause deterioration and, finally, failure of the process.

Actually, as far as the performance of the final tanks is concerned, a most significant factor is the volume occupied by the activated sludge solids after settling for short periods. A commonly accepted method for measuring this property is to settle a 1-l. sample of aerator effluent in a cylinder for ½ hr. and to read the per cent of the total volume occupied by the settled sludge. Then rate of formation of settled sludge volume in the final tanks is the mixed liquor flow times this percentage divided by 100. Manifestly, if a short solids detention period is to be attained in the sludge blanket, the return sludge flow should be equal to or greater than the flow of settled sludge.

Sludge Density Index

The relationship of aerator effluent suspended solids concentration to settled cylinder volume is called the sludge density index and is expressed as:

$$\text{Sludge Density Index} = \frac{\text{aerator susp. sol. conc. (p.p.m.)}}{100 \times \text{settled volume (\%)}}$$

Physically, the sludge density index is the concentration in per cent solids that the activated sludge will assume after settling for ½ hr. Based on the above expression, it can be seen that at a given sludge index the settled cylinder volume will vary directly with the aerator effluent concentration.

For a return sludge rate of 25 per cent of the sewage flow, the maximum permissible cylinder volume is 20 per cent, calculated from the relationship:

$$\text{Per cent settled volume} = \frac{\text{return sludge flow}}{\text{sewage} + \text{return sludge flow}} \times 100$$

At various aerator effluent concentrations, the indices at which the settled cylinder volume will be 20 per cent are:

Aerator Effluent Concentration	Sludge Density Index
800	0.4
1,000	0.5
1,500	0.75
2,000	1.0

Thus, for a given aerator effluent concentration, there is a critical value of the sludge density index below which the formation of settled sludge in the final tanks will exceed the return sludge rate. If the index decreases below this value and an attempt is made to restore final tank balance by reducing aerator effluent concentrations, the sludge age is correspondingly lowered. If the sludge age becomes less than its critical minimum, the process will fail on the basis of insufficient sludge age.

From these considerations it appears that attempts to reduce substantially the volume of an aerator, operated in the conventional manner with less than 6-hr. detention, will usually fail. Furthermore, significant increases of load on the 6-hr. aerator must be met with a corresponding rise in aerator suspended solids concentration in order to maintain ade-

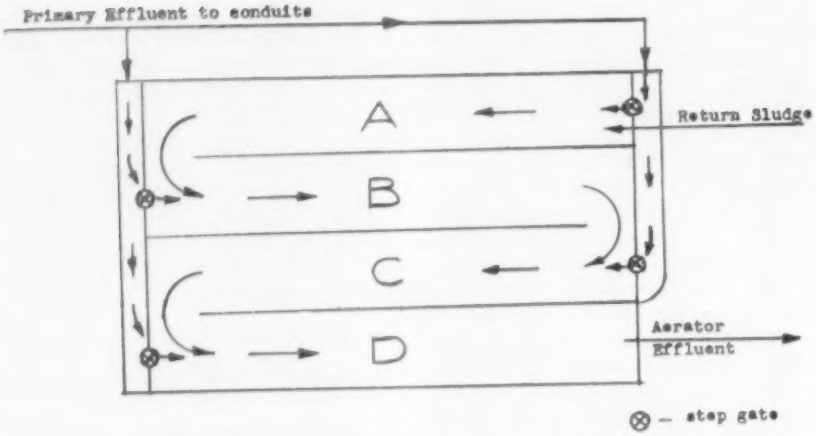

FIGURE 2.—Flow diagram of a step-aeration aerator.

quate sludge age. This imposes a correspondingly higher burden on the final settling tanks. A conventionally operating activated sludge plant is thus limited in its ability to accommodate overload.

Step Aeration

In order for an activated sludge system to operate successfully, there must be provisions for storing enough culture in the aeration tanks to provide adequate sludge age while maintaining low suspended solids concentration entering the final settling tanks. A method of operation has been devised, called step aeration, which permits the use of aeration tanks of one-half the conventional size while still satisfying the foregoing criteria. This was accomplished by providing facilities for adding portions of the primary effluent along the course of the flow of the return sludge through the aerator. A practical application of this principle to a four-pass aeration tank is shown in Figure 2. The return sludge enters at the beginning of the aerator and the primary effluent may be added by means of the "step gates" to the various passes, marked A, B, C and D in the direction of flow.

For a 3-hr. aerator (sewage plus 25 per cent return flow) and a primary effluent containing 90 p.p.m. of suspended solids, a 3.5 day sludge age will require an average concentration of 2,000 p.p.m. of solids in the aeration tank. If the aerator is operated as a conventional unit, each pass will contain 2,000 p.p.m. of solids and will circulate this concentration through the final tanks. By using the step gates for incremental addition of primary effluent, the aerator effluent concentration may be markedly reduced (Figure 3) while maintaining the same sludge age.

As sewage additions are shifted towards the aerator outlet, culture is stored under aeration at higher concentrations toward the inlet, and the concentration of the aerator effluent is reduced. Thus conventional operation (Mode 1) requires that the sludge density index must equal or exceed 1.0 in order to limit settled sludge volume to a maximum of 20 per cent and favor stable operation. On the other hand, by shifting sewage additions to the C–D pass (Mode 4), settled sludge volumes would be limited to the critical 20 per cent as long as the sludge density index is above 0.35. Because of this facility, step aeration

can accommodate the normal range of index with a 3-hr. aerator (detention computed on conventional flow pattern). Moreover, by sacrificing flexibility and confining sewage additions to the C–D pass, the step aerator may operate at 150 per cent of the normal sewage load as long as the sludge density index exceeds 0.52 (based on an average aerator concentration of 3,000 p.p.m. suspended solids and aerator effluent of 1,040 p.p.m.).

Choice of Additions

In the operation of any step aerator, the first consideration is the daily raw suspended solids load in the primary effluent. This quantity is multiplied by the chosen sludge age to determine the weight of activated sludge solids required in the aeration tank. For a given tank volume, the average aerator solids concentration may then be calculated. In order to limit cylinders to a maximum of 20 per cent at a sludge density index more than 0.5, the aerator effluent solids concentration should not exceed 1,000 p.p.m. The ratio of average aerator solids concentration to aerator effluent solids concentration will then dictate the mode of addition. Some primary effluent may be added to the first pass of the aerator when this solids ratio is

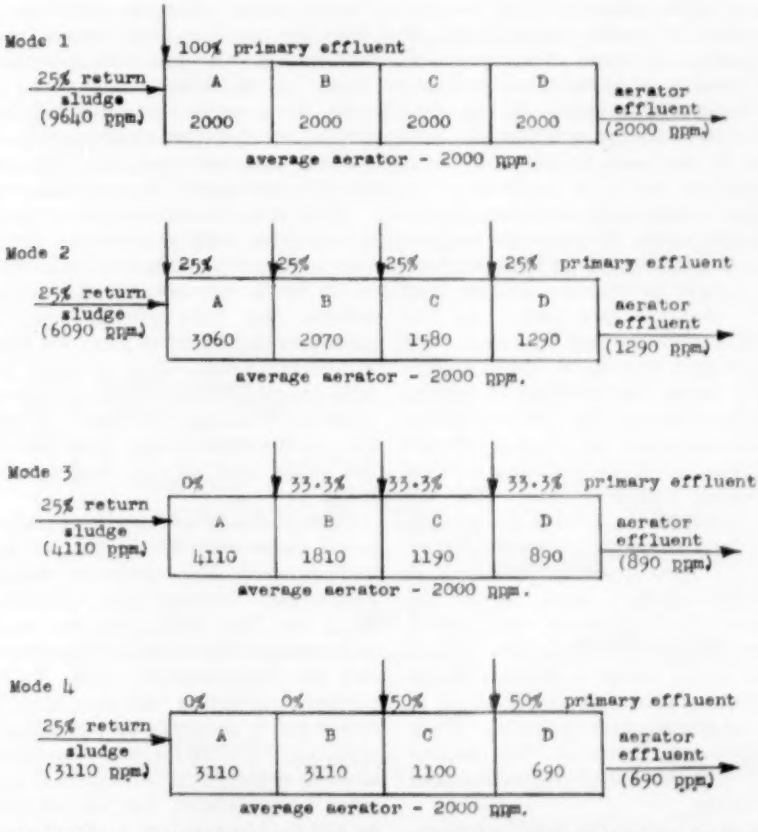

FIGURE 3.—The influence of different modes of sewage addition on aerator concentrations at constant sludge age.

less than 1.55 (2,000/1,290) (Figure 3, Mode 2). As this ratio increases, it is necessary to completely close the A pass gate. For a solids ratio of about 2 to 1, B and C pass addition, or B, C and D pass addition, may be employed. As the ratio approaches 3 to 1, it is necessary to close the B as well as the A gate, thus adding primary effluent to only the C and D passes. The solids concentrations for any combination of step gate additions may be calculated mathematically (see Appendix).

Control of Solids

In actual plant operation, however, the average aerator solids concentration is determined daily by analysis of a grab composite sample from all the passes. The rate of wasting activated sludge is adjusted to maintain the chosen sludge age. At the same time, settled sludge volumes are kept below 20 per cent by the use of an appropriate mode of addition. In practice, in order to overcome higher final tank solids loading due to peak flows, it is desirable to limit the settled sludge volumes to 15 per cent rather than 20 per cent. As the settled sludge volume increases over 15 per cent due to a lower sludge density index, the addition of sewage is shifted toward the aerator outlet in order to lower the aerator effluent solids concentration and hence limit the settled sludge volume. On the other hand, if the settled sludge volumes drop below 8 per cent for any sustained period, sewage addition is gradually shifted away from the aerator outlet in order to promote better clarification from this denser, higher index sludge. These changes usually are made on the basis of a trend of several days duration. When conditions are favorable, the position of the step gates may not be changed for months.

Experience with step aeration operation has been confined between the limits of Modes 2 and 4 (Figure 3), which may be considered rather mild to rather severe "stepping" of the flow addition. Within these limits it has been possible to satisfy the oxygen requirements of the system with the installed air diffusion capacity. In the control of frothing it has been helpful to add at least some primary effluent to the last pass of the aerator, where froth production is usually most acute.

Air Supply

The rate of air supply is adequate when controlled to maintain from 2.5 to 5.0 p.p.m. of dissolved oxygen at the aerator outlet. Major changes in the air supply rate are made on the basis of the dissolved oxygen trend rather than on individual determinations. Some consideration may be given to lowering the air supply rate during the low flow night period. At some plants, however, the rate may remain unchanged for months.

With step aeration it has been unnecessary to vary appreciably the return sludge flow. Return sludge rates of 25 to 35 per cent of the average sewage flow have proven adequate. Once established, these rates are maintained for years. This is made possible by controlling settled sludge volumes by changing the mode of addition to the aerator and thus limiting the solids load on the final settling tanks.

The operation of step aeration depends upon units of the proper type and capacity for optimum results. Preliminary settling tanks, providing about one hour detention, are useful in lowering over-all raw sludge volume and air requirements. This is important considering that step aeration makes heavy demands on air diffusion capacity. Air ratios are comparable to those required in conventional activated sludge plants, but the air must be supplied to aeration tanks of about one-half conventional capacity. A dif-

fusion system capable of supplying these higher air demands per unit aerator volume should be provided.

Final Settling Tanks

Previous discussion has indicated the importance of final settling tanks to the activated sludge process. These tanks must not only produce an effluent of acceptable quality at reasonable overflow rates but must separate and deliver a fresh activated floc to the return sludge pump. These criteria can be satisfied by the "Gould" type final tank which utilizes the phenomenon of sludge density currents, as shown in Figure 4.

Type 1 tanks have been utilized in sewage treatment plants served by sanitary sewer systems. The Type 2 tank is a means of accommodating the effects of varying salt water infiltration caused by tidal action on combined sewer systems. A decreasing density of liquor entering the final tanks causes currents to rise near the inlet and carry solids to and along the water surface. The outlet weirs are placed as far as possible from the influent end of the tank in order to minimize solids carryover by these currents. The common feature of both types of tanks is the introduction of aerator effluent near the tank floor and the flight action in the same direction as the density flow of the sludge to the return sump.

Sludge Thickener

In order to maintain the desired sludge age, activated sludge solids must be removed from the system at the same rate as they are being formed. To keep the volume of the removed excess sludge solids to a minimum, separate thickening tanks are often provided. These tanks receive the dilute sludge and concentrate it in preparation for disposal. Circular thickening tanks equipped with picket fence mechanisms have been suitable for this purpose at loading rates up to 4 lb. per sq. ft. per day of activated sludge solids. At rates of addition of aerator effluent of approximately 500 gal. per sq. ft. per day of thickener, the effluent will be of acceptable quality. The thickening of return sludge alone, however, has given rise to some difficulty, inasmuch as long liquid de-

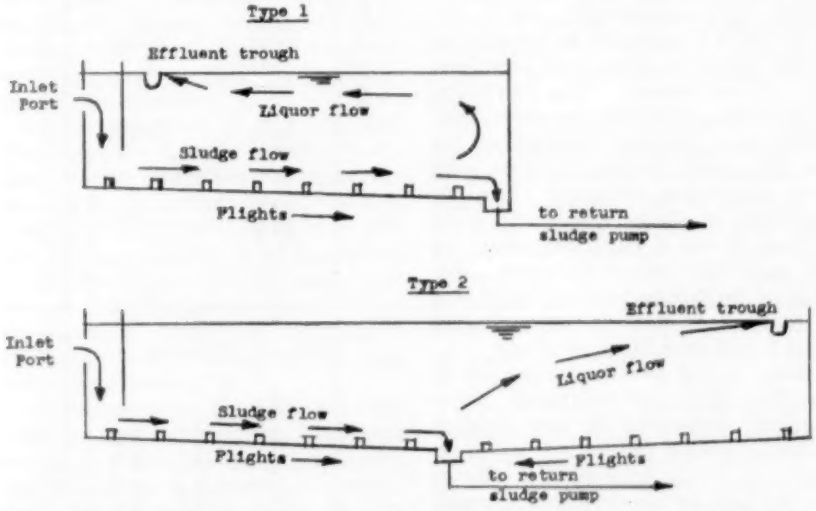

FIGURE 4.—Diagrammatic representation of the "Gould" type final settling tanks.

tentions resulted in septicity and some loss of sludge concentration.

Discussion

As has been previously stated, aeration tanks with facilities for step flow addition have accommodated higher loadings when dissolved oxygen, settled sludge volumes and return sludge flow have been held within given limits. Similarly, operation has demonstrated that sludge age must also be kept within certain limits for favorable results. For the conditions encountered; namely, the treatment of domestic sewage from a community with somewhat higher than normal water consumption, the lower limit of sludge age may be taken as three days. Any attempt to operate at substantially below this limit for a period exceeding a few days results in the formation of a voluminous floc with poor settling qualities and a low sludge index. Usually this overloaded sludge is characterized, microscopically, by large numbers of filamentous organisms.

The upper limit of sludge age appears to exhibit some tolerance based on local conditions. Operation at sludge ages substantially above the upper limit is indicated by peptization of the floc and consequent deterioration of the effluent. This starvation of the sludge, if allowed to persist, may result in a complete breakup of the floc into small discrete particles with poor coagulating properties. The upper limit of sludge age seems to be between 4 and 6 days, depending on the particular treatment plant. Because of these considerations, operation is confined to a sludge age ranging from 3 to 4 days for most plants.

Some typical results of treatment at the New York city treatment plants regularly employing the step aeration activated sludge process are shown in Table I. It is believed the effluent quality and air requirements approximate those to be expected from con-

TABLE I.—Typical Results of Treatment by Step Aeration

Treatment Plant	Date	Sewage Flow (m.g.d.)	Return Sludge (%)	Air Ratio (cu. ft./gal.)	Aeration Period[1] (hr.)	Overflow Rate (gal./sq. ft./day)	Sludge Age (days)	Aerator Effl. Susp. Solids (p.p.m.)	Sludge Density Index	Final Effluent		Removals	
										Susp. Solids (p.p.m.)	B.O.D. (p.p.m.)	Susp. Solids (%)	B.O.D. (%)
Wards Island (Battery C)	Apr.–Dec. 1954[2]	50	28	.43	2.5	870	3.4	950	0.9	6	6	95	94
Hunts Point	1954	95	28	.54	2.9	800	3.6	700	0.8	19	16	86	86
Tallmans Is.	1954	31	28	.59	4.2	900	5.5	800	0.8	17	12	88	90

[1] Calculation based on conventional activated sludge flow.
[2] On modified aeration Jan.–Mar., 1954 during piping changes; step aeration remainder of year.

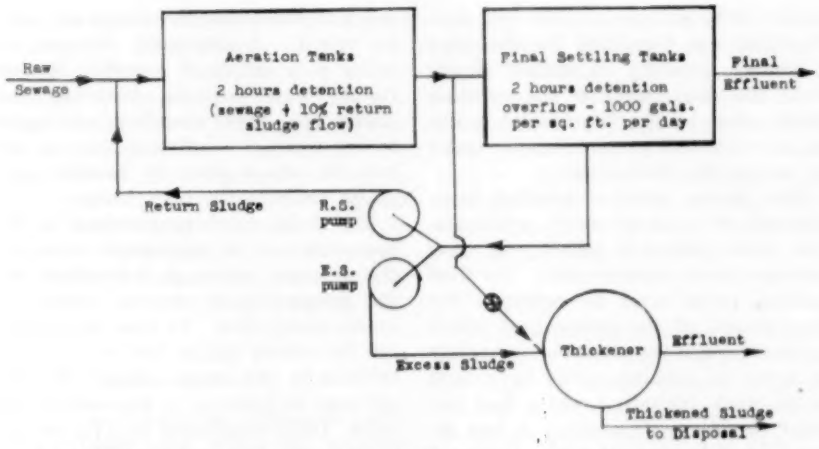

FIGURE 5.—Flow diagram of a modified aeration sewage treatment plant.

ventional activated sludge operation using 6-hr. aeration tanks. In fact, the most favorable results were attained at the Wards Island treatment plant with the lowest aerator detention.

The step aeration process affords full activated sludge treatment, using only one-half the conventionally required aerator volume. In addition to savings in construction cost, there is a significant decrease in site area requirements. This is particularly important in densely populated areas where it may not be possible to obtain sufficient land to build the conventional type treatment plant

In the operation of the activated sludge process the step aeration facilities provide flexibility for meeting changes in the settleability of the floc. Aerator loadings up to 50 lb. of B.O.D. per day per 1,000 cu. ft. of aerator capacity, based on yearly averages, have been successfully accommodated. Some assistance in controlling frothing in the aerators is provided by the step addition of sewage.

Operating costs for both the step aeration and conventional activated sludge processes are essentially equal. Labor charges, compressed air power costs and volumes of sludge for disposal are about the same. These operating costs may only be reduced by utilizing processes affording substantially lower treatment efficiency.

Modified Aeration

For some receiving waters, conditions may require higher removals than can be provided by plain sedimentation, yet may not demand the treatment efficiency and high cost of activated sludge. Based on experimental work there was developed and placed in service a method of obtaining intermediate clarification which is referred to as modified sewage aeration.

It was found that when the sludge age is extremely low, a stable process of biological sewage treatment results. Modified aeration may be adapted to activated sludge plants by providing means for maintaining these low sludge ages, or it may be employed in plants designed for and limited to this procedure. A typical flow diagram of a modified aeration plant is shown in Figure 5.

This process is normally operated with aeration tanks having about 2-hr. detention, based on sewage flow plus 10 per cent return sludge, and final settling tanks overflowing at

about 1,000 gal. per sq. ft. per day. Facilities are furnished for directing a limited quantity of settled sludge from the final tanks to the aeration tanks and to the thickening tanks in any desired proportion, in order to control the sludge age.

The process can be operated satisfactorily without primary sedimentation tanks, although possibly at some increase in air requirements. The final settling tanks must be equipped for the removal of the grease load which passes through the aerators. A variety of types of settling tanks have been satisfactory, including center feed circular clarifiers. Thickening is best accomplished in circular tanks equipped with picket-fence mechanisms. Because of the higher concentration obtained with modified aeration sludge, these tanks may be loaded at the rate of 15 lb. per sq. ft. per day or higher. Some method of diluting the excess sludge prior to thickening may be necessary to maintain overflow rates high enough to avoid septicity.

Sludge Age Control

In the operation of modified aeration, control of the sludge age is essential. If the sludge age is too low, effluent quality is reduced and if it exceeds a critical limit, settleability of the sludge is lowered. Prolonged operation at these higher age levels results in the growth of various organisms associated with bulking sludges and in eventual process failure. The magnitude of this upper age limit is affected by the characteristics of the sewage being treated. Also, as sewage strength and temperature increase, this upper limit of sludge age decreases. If biological stability is to be maintained, the operating sludge age must be reduced below that limit.

A determination used by the plant operator for process control is the sludge density index. An index of about 2.0 indicates operation close to the correct age range, while a higher index signifies that the sludge age may be raised. A continued decrease in index is a signal of possible process failure and requires lowering the sludge age. These considerations apply to dry weather conditions only, as silt brought into a plant by rainfall produces temporarily high indices.

The sludge age is proportional to the concentration of suspended solids in the aeration tanks as determined by the proportion of return sludge to excess sludge flow. To raise the sludge age the return sludge flow is increased relative to the excess sludge and the age may be lowered by decreasing this ratio. Once established at a particular sludge age, the system will tend to maintain that age provided the removal efficiency does not change materially.

Sludge ages for modified aeration lie in the range of 0.2 to 0.5 days. The aeration tanks, therefore, contain relatively small quantities of active material. In the event that deleterious organisms develop, as manifested by significant decreases in sludge density index, it thus becomes possible to completely remove this small quantity of poor culture and replace it with new modified aeration sludge within a day. This is accomplished by shutting down the return sludge pump and sending all solids in the final tanks to the thickeners. Within a short time, straight sewage aeration is being practiced. When all the undesirable culture has been removed from the aeration and settling tanks, the return pumps are started. The return to excess sludge flow ratio is then set to produce a somewhat lower age, and within several hours normal modified aeration treatment is re-established. This procedure is usually necessary only when sewage strength or temperature changes are significant, as with seasonal fluctuations or certain shock loads.

Inasmuch as modified aeration does not call for large masses of culture in the aeration tanks, air require-

ments respond markedly to the diurnal changes in sewage load. In this connection, it has been found practical during the night (possibly from midnight to 10 AM) to reduce the air supply rate by as much as one-third to one-half of the daytime rate. Due care must be exercised to keep air rates high enough to avoid difficulties from diffuser clogging. In general, a dissolved oxygen of 2 p.p.m. at the aerator outlet has been a satisfactory minimum.

Solids Concentration

The considerations discussed previously apply also when the modified aeration process is adapted to an activated sludge treatment plant, either conventional or step aeration. A fundamental difference, however, is that the primary effluent will be subjected to secondary treatment rather than raw sewage. Because of this, the concentration of suspended solids in the aeration tanks must be considerably lower to satisfy the sludge age requirements.

The larger aerators introduce various problems of operation. In order to control sludge age, aerator solids concentrations are lowered to a point where the range of tolerance may be 25 p.p.m. The lower air requirements per unit of aerator volume may reduce diffuser loadings below critical levels for proper air distribution. Attempts to reduce aerator detention time by removing units from service are limited by the flow capacity of the connections to and from the aerators.

Another problem associated with low aerator solids concentrations is that of wasting the excess sludge. The large return sludge pump capacity means correspondingly dilute return sludge. Large proportions of this return sludge flow must be sent to the thickeners, presenting a problem of high liquid loading there. Considerable attention must be directed to reducing return sludge rates to 5 to 15 per cent, if thickener operation is to to be practical. Where aerator detention can be decreased to 3 hr. or less, it has been feasible to adapt modified aeration to activated sludge plants. In these cases, treatment efficiencies have been comparable to those from modified aeration plants without primary settling tanks.

Efficiencies

Treatment efficiencies, as well as other salient operating data, are shown in Table II, which illustrates typical results at six sewage treatment plants treating approximately 316 m.g.d. of sewage by the modified aeration process. Three sewage treatment plants employ primary sedimentation prior to aeration and the other three aerate raw sewage.

Air demands for modified aeration range from 0.3 to 0.5 cu. ft. per gallon, with perhaps somewhat higher requirements where primary sedimentation has been omitted. Aerator detentions vary from 1.5 to 3 hr. without any apparent influence on treatment efficiency.

Effluent quality varies from 34 to 50 p.p.m. of suspended solids and from 32 to 64 p.p.m. of B.O.D. Plant efficiencies range from 64 to 81 per cent removal of suspended solids and 57 to 76 per cent removal of B.O.D. With these process efficiencies, modified aeration lies about halfway between plain sedimentation and activated sludge.

Discussion

It has been observed that treatment of sewages stronger in B.O.D. results in somewhat lower removal efficiencies. This is especially true where a major portion of the non-settleable B.O.D. is soluble. An example of this is the Bowery Bay sewage treatment plant where the primary settling tanks remove only 17 per cent of the B.O.D. An additional 40 per cent of the

TABLE II.—Typical Results of Treatment by Modified Aeration

Treatment Plant	Date	Sewage Flow (m.g.d.)	Air Ratio (cu. ft./gal.)	Aeration Period (hr.)	Overflow Rate (gal./sq. ft./day)	Sludge Age (days)	Aerator Eff. Susp. Solids (p.p.m.)	Sludge Density Index	Final Effluent		Removals	
									Susp. Solids (p.p.m.)	B.O.D. (p.p.m.)	Susp. Solids (%)	B.O.D. (%)
With Primary Settling												
Wards Is.	Apr.–Dec. 1954	100[1]	.30	3.0	880	.24	130	2.2	39	39	68	61
26th Ward	1954[2]	40	.54	2.6	670	.37	280	1.4	34	38	76	71
Bowery Bay	May–Aug. 1954[3]	37	.39	1.5	930	.17	200	1.6	50	64	64	57
Without Primary Settling												
Owl's Head	1954	83	.48	2.1	770	.21	370	1.7	44	59	74	63
Jamaica	1954	45	.44	1.7	730	.49	1,000	2.1	38	35	81	76
Rockaway	1954	11	.38	2.1	850	.44	470	2.4	38	32	67	64

[1] Batteries A and B.
[2] Full months of modified aeration operation (excluding summer period of step aeration).
[3] Operation with one aeration tank.

B.O.D. in the raw sewage is removed by secondary treatment, resulting in an over-all removal of 57 per cent.

Sewage at the Owl's Head sewage treatment plant also has a relatively high soluble B.O.D., averaging 67 p.p.m. These two plants require low sludge age and precise control for effective treatment. On the other hand, the Rockaway sewage treatment plant, treating a raw sewage with a low soluble B.O.D. of 29 p.p.m., can tolerate a wider range of sludge age. The other treatment plants, listed in Table II, fall between these extremes.

Where the sewage is readily amenable to treatment by modified aeration, favorable results may be secured for extended periods. Thus, at the Jamaica sewage treatment plant modified aeration has been utilized since 1942 with results similar to 1954 operation.

An attractive feature of the modified aeration process is the compactability of the excess sludge solids. Whereas activated sludge solids can be thickened to a concentration of only about 2 to 3 per cent total solids, modified aeration sludge will reach an average of more than 4 per cent for treatment plants with primary settling. For modified aeration plants without primary tanks, effective thickening has produced sludge concentrations averaging about 7 per cent. As a result of these higher concentrations for modified aeration, the volume of sludge for disposal is materially less than for activated sludge. Where both processes have been operated at the same plant, total sludge volume produced by operation of the modified aeration process has been roughly one-half that from activated sludge.

Comparative operation has also shown that, although the quantity of solids removed from sewage by modified aeration is less than for the activated sludge process, gas production is greater from the digestion of the sludge produced by modified aeration. This is the result of loss of gasifiable solids by aerobic digestion, which is appreciable in the activated sludge process.

In comparing the construction and operation costs of the modified aeration plant (without primary tanks) and the step aeration activated sludge plant, consideration should be given to the following factors:

1. Modified aeration requires about two-thirds of the total tank volume for sewage treatment.
2. Lower sludge volume will reduce sludge disposal costs.
3. Lower air ratios are accompanied by a reduction in power costs.
4. The higher digester gas yield from sludge digestion, in conjunction with the lower air ratios, presents an attractive situation for providing gas engines and making the plant independent of outside power sources.
5. The area of the treatment plant site is reduced to the minimum for a secondary treatment plant.

It must be recognized, however, that these favorable aspects result from a sacrifice of effluent quality. Where high quality effluents are mandatory, the modified aeration process may prove inadequate.

The data of Table II apply to medium strength domestic sewage. Furthermore, experience indicates that stronger sewages require a reduction in sludge age for stable operation. Because of this undesirable circumstance, it is conceivable that with certain strong or abnormal sewages modified aeration will not produce satisfactory treatment efficiency.

Activated Aeration

Recently a process referred to as activated aeration has been developed and placed in service. This process operates in conjunction with the activated sludge system to extend its flexibility and to obtain varying degrees

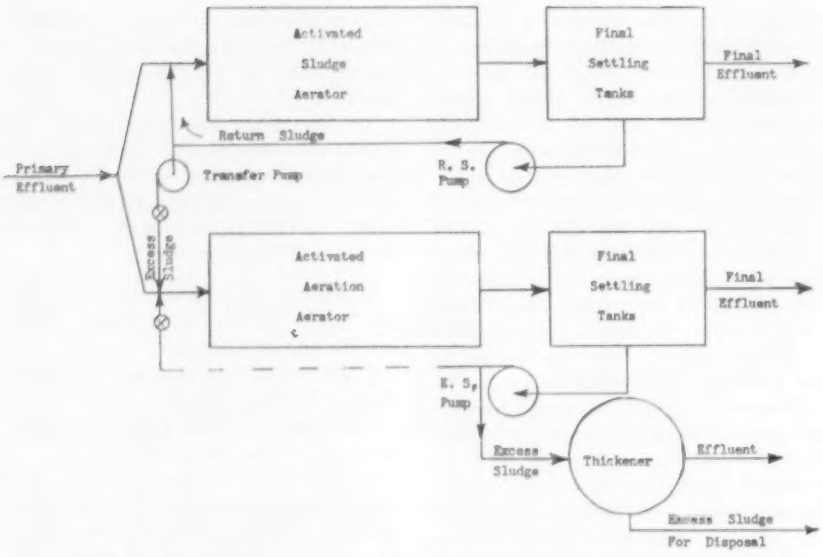

FIGURE 6.—Flow diagram of an activated aeration sewage treatment plant.

of partial treatment and lower operating costs.

The process requires an activated sludge plant, conventional or step aeration, whose secondary system may be separated into independent sections (Figure 6). One section, using the activated sludge process, produces fresh excess activated sludge culture which is transferred to the other section for re-use in clarifying additional sewage.

The culture produced in the activated sludge section which is ordinarily wasted to sludge thickening tanks, is transferred to the activated aeration section. This dosage of activated sludge solids depends on the quantity of activated sludge produced and the flow to the activated aeration section. For equal division of flow between the two sections, this may amount to approximately 75 p.p.m. of solids. The primary effluent is aerated in the presence of these small quantities of activated sludge. The aerated liquor is then subjected to settling in the final tanks where settled solids are removed for thickening, while the clarified effluent overflows.

During two years of plant scale operation, the excess activated solids in the form of return sludge were transferred from the step aeration system by means of a pump. The discharge was controlled to maintain the desired sludge age in the section devoted to the activated sludge process. The transferred solids were added to the primary effluent entering the activated aeration tanks and the mixture was aerated for slightly less than 3 hr. The air supplied was close to the minimum for maintaining the aerator solids in suspension. The results of this co-ordinated operation of activated aeration and its step aeration source of activated sludge are shown in Table III.

Efficiencies

Based on 75 p.p.m. of suspended solids removed from 49 m.g.d. of flow by step aeration, the 53 m.g.d. of primary effluent entering the activated aeration battery was dosed with 69

TABLE III.—Treatment Results Activated and Step Aeration [1]

	Activated Aeration	Step Aeration	Combined Treatment
Flow treated (m.g.d.)	53	49	102
Aer. det. (hr.)	2.9	2.5	2.7
Air ratio (cu. ft./gal.)	.22	.52	.36
Final tank overflow (g.p.d./sq. ft.)	1,000	860	—
Susp. sol. (p.p.m.)			
Aer. effl.	220	1,000	—
Return sludge	1,100[2]	3,900	—
Thickened sludge (%)	—	3.3[3]	3.3
Process effl. (p.p.m.)			
Susp. sol.	33	7	21
B.O.D.	33	6	20
Removal (%)[4]			
Susp. sol.	75	95	84
B.O.D.	71	95	82

[1] Wards Island, Mar., 1953 to Jan., 1955, except Jan.–Mar., 1954.

[2] Settled sludge to thickener.

[3] 1953 only. During 1954 combined thickening practiced.

[4] Based on raw sewage strength of 133 p.p.m., suspended solids and 113 p.p.m., B.O.D.

p.p.m. of activated sludge culture ($49/53 \times 75$ p.p.m.). The mixture was aerated for 2.9 hr. and then settled at an overflow rate of 1,000 gal. per sq. ft. per day. The process effluent averaged 33 p.p.m. of suspended solids and B.O.D., having been increased approximately 4 p.p.m. because of sludge thickening. The air supplied averaged only 0.22 cu. ft. per gallon.

The combined processes produced an effluent containing 21 p.p.m. of suspended solids and 20 p.p.m. of B.O.D. Over-all removals of 84 and 82 per cent of suspended solids and B.O.D. respectively were secured using an air ratio of 0.36 cu. ft. per gallon at an aerator detention of 2.7 hr.

The excess sludge from the process was concentrated to 3.3 per cent total solids in two rectangular final settling tanks. These improvised thickeners often showed signs of solids overloading by loss of effluent quality and solids concentration. In 1954 these tanks were further required to concentrate primary solids according to the combined thickening process. The activated aeration effluent shows the effect of thickening all the secondary solids from both the activated aeration and step aeration batteries plus, in 1954, the additional effects of thickening primary solids.

Some opportunities arose to investigate the possibilities of improving removal efficiency of activated aeration. The process was operated for one month using 4.4 hr. of aerator detention at an air ratio of 0.29 cu. ft. per gallon. This higher air ratio was necessitated by a 50 per cent increase in aeration tank volume. The effluent contained 24 p.p.m. of suspended solids and B.O.D. This one-third improvement in effluent quality was attained at the cost of some increased air and loss of thickened sludge concentration.

Later, when the normal quantities of activated sludge were transferred to one-half the usual activated aeration flow, thereby doubling the relative dosage of active solids but maintaining the same aerator detention time, a significant improvement in effluent quality was noted over normal operation. For the three weeks of this trial, the effluent averaged 23 p.p.m. of suspended solids and 23 p.p.m. of B.O.D., including 8 p.p.m. added by sludge thickening.

Features

The activated aeration process has been capable of providing an effluent of intermediate quality at savings in air compression power costs when compared with the activated sludge process. Since it was not necessary to recycle active solids through the activated aeration system, deleterious cultures were not proliferated and the process exhibited excellent stability. The difficulties of control experienced with modified aeration at some locations were avoided. Some reductions in sludge volume as compared to the

activated sludge process alone were also realized. Finally, where plant layout permits, activated aeration operation should afford complete flexibility of operation between the range of full activated sludge and straight sewage aeration, so that plant effluent quality may be adjusted to meet the requirements of the receiving stream.

Recent Trends

Some changes in conditions affecting activated sludge treatment have been observed during the past few years. Such gradual changes, occurring over a period of years, are difficult to assess, especially when marginal effects are being considered.

These marginal effects indicate a reduction in process stability. Thus, for one step aeration activated sludge plant the sludge density index for the period 1940 to 1949 averaged 1.6. During the following four years this average was reduced to 1.1. For the year 1954 the index was further reduced to 0.8. These last values were obtained with aerator loadings lower than for the 1940–1949 period.

As these yearly averages have decreased, the indices developed at the lower sewage temperatures between December and March have recently begun to approach critical levels. The situation has been met, so far, by adjusting sludge age nearer to optimum values and attempting to maintain these levels. Tolerance of the activated sludge process to operation at higher sludge ages has been sharply reduced. Any significant extension of this trend may lead to serious consequences.

A parallel effect has been observed at some modified aeration plants. The consequent lowering of the sludge age in order to maintain process stability here results in sacrifice of treatment efficiency.

These influences on the activated sludge process have become apparent with the increase in the sale and use of synthetic detergents. A natural question is whether or not some element or type of detergent compound is adversely affecting biological sewage treatment. Some papers have appeared in the literature discussing the effects on the biology of the activated sludge flora. It should be noted, however, that if coagulability of the floc is reduced and its dispersion increased, this might account for the observed changes. Investigations directed towards clarifying these questions and comparing operating experience on a national scale should prove beneficial.

APPENDIX

Sludge Age

Sludge age has been defined as the weight of the dry suspended solids in the aeration tanks divided by the weight of the dry suspended solids in the sewage or primary effluent added daily. It may be calculated from the expression,

$$\text{Sludge age (days)} = \frac{V \times A}{Q \times C}$$

where V is the aerator volume in million gallons, A is the average aerator concentration of suspended solids in p.p.m., Q is the sewage flow in m.g.d., and C is the sewage or primary effluent suspended solids in p.p.m.

Where the volatile content of the activated sludge is subject to radical changes, it has been proposed to substitute the weight of volatile suspended solids in the aerator rather than the total. Furthermore, where soluble industrial wastes are being treated, it has been proposed to determine load based on B.O.D. However, when treating normal domestic sewage, the expression as given has proved adequate.

Solids Concentration in Step Aeration

For calculating the suspended solids concentrations of the aerator, or return sludge, the following expressions have been derived:

Return sludge concentration,

$$C_R = \frac{(Q_S + Q_R)C_A - Q_S C_P}{Q_R}$$

Per cent return sludge,

$$r = \frac{Q_R}{Q_S} \times 100 = \frac{C_A - C_P}{C_R - C_A} \times 100$$

Pass concentration,

$$C_X = \frac{Q_R C_R + Q_X C_P}{Q_R + Q_X}$$

where

C_A = aerator effluent suspended solids concentration, p.p.m.

C_P = primary effluent suspended solids concentration, p.p.m.

C_R = return sludge suspended solids concentration, p.p.m.

C_X = suspended solids in the pass, p.p.m.

Q_R = return sludge flow, m.g.d.

Q_S = total primary effluent flow, m.g.d.

Q_X = total primary effluent flow added up to and including the point X, m.g.d.

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3. Torpey, Wilbur N., "Practical Results of Step Aeration." *Sewage Works, Jour.*, **20**, 5, 781 (Sept., 1948).
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5. Gould, Richard H., "Sewage Aeration Practice in New York City." *Proc. Amer. Soc. Civil Eng.*, **79**, Separate No. 307 (Oct., 1953).
6. Chasick, Abraham H., "Activated Aeration at Wards Island." THIS JOURNAL, **26**, 9, 1059 (Sept., 1954).

PROFESSIONAL OPPORTUNITY

The city of Atlanta, Ga., is accepting applications for a position described as professional chemistry work of a specialized nature in interpreting chemical tests made in sewage treatment plants, and related work as required.

Salary range is $386—$476 per month. A college graduate in chemistry or chemical engineering is preferred. Further information may be obtained from Carl T. Sutherland, Director of Personnel, City of Atlanta, Personnel Department, 4 City Hall, Atlanta, Ga.

DISINFECTION IN MILITARY SEWAGE TREATMENT PLANTS *

By Harold A. Thomas, Jr.

Associate Professor of Sanitary Engineering, Division of Engineering and Applied Physics, Harvard University, Cambridge, Mass.

Disinfection in sewage treatment practice ordinarily is accomplished by the addition of elemental chlorine to sewage by vacuumatic or diffusion type dosing equipment. In the following discussion, the term "disinfection" will be considered broadly whether accomplished with elemental chlorine, chlorine compounds, or other disinfection chemicals.

Sewage disinfection practice has derived from investigations made 20 to 30 years ago by the U. S. Public Health Service (1), various water pollution control agencies, and other interested agencies, relating to the survival of enteric pathogens in sewage treatment and their persistence in streams, and from technological developments of equipment relating to efficiency and control of dosage.

Recent developments accentuate the need for a re-evaluation of coliform bacteria standards and chlorination practice. A nationwide investigation of this nature in being planned by the USPHS at the Taft Sanitary Engineering Center, Cincinnati, Ohio.

With the substitution of synthetic detergents for soap, and with the addition of many other new components, the mineral and organic substance of sewage has been altered significantly in the past decade. There is increasing evidence that this has affected treatment efficiency, disinfection and die-away rates.

The amperometric titrator makes possible more precise and sensitive measurements of bacterial chlorine residuals. New laboratory tools, such as the membrane filter, increase materially the efficiency of determining bacterial densities and characteristics. Recent laboratory and field tests indicate the strong possibility that more effective and economical chlorine applications may be obtained by multiple-point dosing in contact tanks properly designed to control mixing and short-circuiting.

There recently has been an increasing recognition of the value of chlorination in aspects of sewage treatment practice other than disinfection.

Modern water treatment methods, particularly those pertaining to effective coagulation, settling, and disinfection, have undoubtedly increased the ability of the modern water treatment plant to treat safely sources of higher coliform density than formerly were deemed suitable for raw water supply.

Pending the results of the re-evaluation survey by the USPHS, the following discussion of chlorination is offered as a guide for military sewage treatment practice in continental United States and overseas installations.

Recommendations

Compliance with Local Rules

Disinfection practice at military sewage treatment plants should comply

* Prepared for and approved by the National Research Council, Division of Medical Sciences, Subcommittee on Waste Disposal, Herbert M. Bosch, Chairman, and approved December 14, 1954, by the parent Committee on Sanitary Engineering and Environment, Abel Wolman, Chairman. Prepared in response to a request from the Office of the Surgeon General of the Army.

at least with requirements relating to streams or effluents promulgated by local water pollution boards or health agencies in the United States and territories having jurisdiction. In the Addendum is presented a discussion of a method of determination of appropriate degrees of disinfection that may serve as a guide to judgment in military posts beyond continental limits or in areas with inadequate or unsuitable disinfection regulations.

Objectives of Chlorination

Chlorination at military sewage treatment plants should be employed only for certain specific objectives. In the absence of these objectives, the use of chlorine is not recommended. The following outline relates to various functions of chlorination.

A. Among the proper uses of chlorine in sewage treatment are the following:

1. Protection of receiving waters, depending on use:
   (a) Water supplies.
   (b) Recreation (bathing) and training.
   (c) Shellfish culture.
   (d) Irrigation.
2. Facilitation of sewage plant control:
   (a) Control of ponding or clogging of trickling filters (intermittent or seasonal application).
   (b) Regulation of activity of return activated sludge.
   (c) Reduction of rate of decomposition during sludge thickening.
   (d) Reduction (upon occasion) of immediate oxygen demand of digestor supernatant returned to plant influent.
   (e) Grease removal (in special circumstances only).
3. For other purposes:
   (a) To prevent disintegration in long, flat, concrete sewers from oxidation of hydrogen sulfide.
   (b) To avoid excessive septicity that will impair "treatability" of sewage in biological units.
   (c) To avoid excessive odor that may create an atmospheric nuisance in the vicinity of the treatment plant.

B. Uses not normally considered justifiable except as may be incidentally accomplished by chlorination for other purposes:*

1. Odor abatement as a substitute for "good housekeeping" in plant maintenance.
2. Reduction of biochemical oxygen demand.
3. Control of aquatic growths in receiving waters.

Research Program

A research and developmental program should be instituted to evolve a type or types of chlorination equipment suitable for emergency, seasonal and stand-by use at military sewage treatment plants.

The objectives of the program should include:

1. Development of rugged and relatively inexpensive dosage devices capable of resisting deterioration during long periods of stand-by operation.

2. Establishment of safe, general rules relating to routine and emergency chlorination practice to serve as an operational guide.

* It is recognized that in emergencies and in special circumstances (as when a water supply contains a high sulfate concentration or a sewage has unusual waste components) the use of chlorine for the control and odor and other nuisances or the reduction of B.O.D. may be indicated.

The importance and merits of chlorination for emergency, and in some cases for routine, use in waste water treatment at military installations should receive additional stress, and accordingly the foregoing program should be given a high priority.

After completion of this program and of the re-evaluation survey previously referred to, consideration should be given to possible modification of disinfection practice at military sewage treatment plants.

ADDENDUM

Calculation of Disinfection Requirement

In municipal practice, disinfection of primary or secondary plant effluents commonly amounts to 98 to 99.8 per cent destruction of coliform bacteria. This is usually accomplished by chlorination to a residual of 0.1 to 0.5 p.p.m. with 15 to 30 min. contact. However, the bacterial reductions needed and obtained vary widely from plant to plant. In the absence of regulations or adequate guidance from local civilian health agencies having jurisdiction over the receiving waters, calculations of the following type may be helpful in establishing reasonable chlorination practice at military sewage treatment plants.

In assessing bacterial removal efficiency in plants or die-away in streams in relation to ultimate water use, the concept of the *decontamination factor* has utility. While in the past it has been customary to report bacterial removals as percentages, such as 99 or 99.9 per cent, these numbers are not easily combined with corresponding values for subsequent purification in streams or water treatment plants. Moreover, since these values are nearly equal to 100, they fail to emphasize the degree of treatment actually attained in respect to health hazards associated with the use of the effluent. The decontamination factor provides this emphasis; it is defined by the following formula:

$$F = \log \frac{100}{100 - P}$$

where P is the percentage of bacteria removed by the treatment device. The merit of the decontamination factors is that the over-all performance of a series of treatments (sewage plant, stream and water plant) may be obtained by finding the sum of the decontamination factors for all treatments.

Example: A primary sewage treatment plant receives raw sewage containing 10^7 coliform bacteria per 100 ml. Fifty per cent of the coliform bacteria are removed in settling; 99 per cent of the remaining are destroyed by chlorination to a 0.2 p.p.m. residual in a contact tank having a detention time of 15 min. The plant effluent is discharged to a stream with an average dilution of 10 to 1. The flow time in the receiving stream is 1.5 days to the intake of a water purification plant. Estimate the coliform density in the raw water of the purification plant.

Treatment	P (%)	F
Primary sedimentation	50	0.3
Chlorination to 0.2 p.p.m. residual	99	2.0
Dilution in stream	90	1.0
Die-away in stream[1]	87.5	1.5(0.60) = 0.9
		Total = 4.2

[1] Assuming F = 0.6 per day. Stream decontamination factors for coliform bacteria and *Salmonella typhosa* usually vary from 0.3 to 0.9 per day depending on temperature, short-circuiting, and antibiotic activity of predatory organisms. The decontamination factor for unit contact time is identical with the parameter k in Chick's law, $P = 100\ (1 - 10^{-kt})$, used to formulate results of die-away and disinfection tests.

The coliform density in the stream at the water plant intake will be

$10^7/10^{4.2} = 10^{2.8} =$ 630 coliform bacteria per 100 ml. In order to meet the USPHS Drinking Water Standards of 1 coliform per 100 ml., the water plant would have to achieve an additional decontamination of log $630/1 = 2.8$. This factor is larger than that considered safely attainable by raw water chlorination alone and therefore a filtration installation is indicated. If, however, the treatment of the town water supply consisted of chlorination only, the situation could be met at the sewage plant by carrying a higher residual through the contact tank, by increasing its detention period, or both. To obtain a reduction to 50 coliform bacteria per 100 ml. in the water plant raw water supply, the total decontamination factor would be increased to

$$4.2 + \log \frac{630}{50} = 4.2 + 1.1 = 5.3.$$

In order to estimate the residual that would be needed to accomplish this additional treatment, the following approximate rule may be used: The coliform bacteria decontamination factor attained in a contact tank of fixed detention time increases in direct proportion to the chlorine residual. Therefore, in order to increase the decontamination factor of the system by 1.1, a residual of

$$\frac{2.0 + 1.1}{2.0} \times 0.2 = 0.31 \text{ p.p.m.}$$

should be carried in the sewage plant effluent.

Reference

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FELLOWSHIPS AND GRADUATE ASSISTANTSHIPS

A number of opportunities will be available in 1956 for advance study. Several fellowships and graduate assistantships have been announced recently. Other opportunities will be published as they are received from colleges and universities. Among those presently available are the following:

National Science Foundation Fellowships

The National Science Foundation plans to award 700 graduate and 80 postdoctoral fellowships to individuals planning to take graduate study in the sciences during the 1956–57 academic year. NSF fellowships are awarded in the mathematical, physical, medical, biological, engineering and other sciences. Closing date for graduate fellowship applications is January 3, 1956 (postdoctoral fellowship applications, December 19, 1955). Application materials may be obtained from local colleges or universities or the Fellowship Office, National Research Council, 2101 Constitution Ave., N. W., Washington 25, D. C.

Annual stipends range from $1,400 per year (first year) to $3,400 per year (postdoctoral). The fellowships will also provide the payment of tuition and fees, dependency allowances for married Fellows and limited travel allowances.

University of North Carolina

Graduate assistantships in sanitary engineering or public health are available for work towards a Master's degree. Graduate assistants receive $1,300 per academic year with possibility of summer employment. Assistantships are available for February, June, and September, 1956.

Further information may be obtained from the Department of Sanitary Engineering, School of Public Health, University of North Carolina, Chapel Hill, N. C.

OPERATIONAL RESPONSIBILITY OF THE SEWAGE TREATMENT PLANT DESIGNER *

By Kenneth W. Brown and David H. Caldwell

Consulting Engineers, Brown and Caldwell, San Francisco, Calif.

Consulting engineers engaged in the design of sewage treatment plants appear, in general, to be of two schools of thought regarding their responsibility following completion of construction. One group feels the job is ended when the plant is built, while the other, recognizing the problems still ahead, assumes a continuing responsibility and undertakes to provide necessary additional services. This paper deals primarily with the latter type of thinking and attempts in a limited way to demonstrate not only the need but the mutual benefit, both to an engineer and the community he serves, of providing assistance and advice during at least the early stages of operation.

The development of a plan for plant operation should not be a last minute affair; something the designer dreams up when construction is virtually complete. On the contrary, operation should be planned from the start of design and should be thought about constantly in developing the design concept and preparing the construction drawings and specifications. Regardless of how he feels about his responsibility after the plant is built, a competent engineer strives from the beginning to provide for efficient, properly integrated, and low cost operation. He is fully aware at the same time of the basic fact that each unit in the treatment process must be designed not only to serve its intended function but to meet all requirements with respect to capacity, hydraulic efficiency and provision for future expansion.

In the opinion of the authors, every engineer who undertakes to design a sewage treatment plant should himself have spent some time as a plant operator or should have on his staff someone who has served in that capacity. There is no substitute for this type experience. On the other hand, competence in design requires something more than a broad background of experience. It is necessary, in addition, to be endowed with a certain innate quality, best described perhaps as an instinctive capacity for creative effort.

Initial Supervision

It seems strange in a way that anyone should even question the need for at least initial help in the operation of a new waste treatment plant. If the design engineer is not so inclined, or if he is not called upon to lend a hand during the early operating stages, there is bound to be some confusion and a resulting tendency toward fault finding and adverse criticism. In such an event, even the most carefully and intelligently designed facilities may, through ignorance, fail to function at full efficiency.

Rarely will a new treatment plant operate at top efficiency without any help on the part of the engineer. For example, in communities, where an old plant is being replaced or a second plant is being constructed, an existing staff of trained attendants may be

* Presented at 1955 Annual Meeting, California Sewage and Industrial Wastes Assn.; Riverside, Calif.; April 27–30, 1955.

ready to take over and conceivably could do so with little or no assistance from the design engineer. Similarly, a city undertaking sewage treatment for the first time could employ a superintendent with an adequate background of training and experience and thus could get by without any outside help. But these are exceptions and not the rule. In California the consulting engineer generally is expected to provide a service beyond that required for the purposes of design and construction. His clients look to him for needed counsel and guidance during the breaking in period, although it may be an unpaid effort.

Because of the unremunerative nature and other adverse features, some engineers object to giving further service and contend their job is finished when the plant is built. The majority, however, recognize a continuating obligation and make every effort to be helpful during the early stages of operation. Their policy in this respect is governed not by any compulsion in a legal sense but by a feeling of moral responsibility for the successful outcome of their creative efforts. They also feel a compelling desire to justify the confidence imposed in them by the willingness of their clients to accept their judgment. In nearly all cases, the sum of money to be spent for sewage treatment facilities is high in proportion to community income and it is a sobering thought to the engineer who is given the design responsibility.

Limitation of Engineering Service

Failure of some engineers to provide any service beyond that required for design and supervision of construction can be attributed to one or more reasons.

1. The fee schedule suggested by the American Society of Civil Engineers for this type of work extends only to design and makes no provision for initial operation of the completed facility. Therefore, designers are naturally a little reluctant to take on the added and unremunerative job of training plant attendants and checking plant peformance. Experience indicates that many communities, particularly those of limited size, look to the design engineer for initial guidance and seldom expect to pay for this type of assistance.

2. The distance to a job and the cost of getting there can be significant. If the plant is in a relatively remote location and requires substantial travel time on the part of the engineer, he is likely to take a dim view of any attempt to enlist his gratuitous aid on matters of operation. The best solution in a situation of this kind is obviously that of a contract arrangement which includes an adequate allowance for any help that might be needed after completion of construction.

3. It is customary in some states, though not yet in California, for a state agency to furnish direct assistance in starting a new plant. If the state is in a position to lend a hand in training plant attendants, consulting engineers can be expected to react accordingly and to assume that their job is ended when construction is complete.

4. Too much work could be a contributing factor. Nevertheless, the conscientious engineer will still plan his work to meet the needs of his clients and will make no change in the type of service he normally provides.

5. A lack by the designer of operating knowledge and experience. Some engineers occasionally have the temerity to take on a design assignment regardless of how little they know about the intricacies of sewage treatment and sewage treatment processes. But the picture changes when the time comes to operate the completed facilities. Faced then with the complexities of

treatment plant performance and control, engineers in this category find it difficult to be helpful and are not inclined to assume further responsibility.

Incidents of this kind stem primarily from the understandable desire of a city or community to favor the employment of a local engineer. In some cases they result either in a joint venture with a qualified engineer or in an arrangement whereby a sanitary engineer is engaged as a consultant. In other cases, unfortunately, the local engineer fails to obtain expert help and tends instead to look to equipment manufacturers for needed advice.

Scope of Operational Assistance

Stated briefly, the additional duties of the designer during the breaking in period, or possibly for as long as the first year of operation should be as follows:

1. Confer with and assist the client in the selection of a qualified superintendent. In providing this service, names of possible applicants can be suggested, preliminary contacts can be made, and information can be furnished regarding salary scales for similar installations.

Employment of a superintendent and operating personnel should be arranged well in advance of the time the plant is ready to start. The superintendent in particular should be available during at least the closing stages of construction, thus giving him an opportunity to become properly familiar with all of the mechanical and electrical equipment. If difficulty is encountered in finding an experienced superintendent, consideration may well be given to the possibility of employing the resident engineer in charge of inspection. This is especially desirable in the case of young men who have also worked on the design of the plant and are anxious to acquire a background of actual operating experience. Given proper assistance by the consulting engineer or by a consultant employed to advise on plant operation, these men can be counted on, because of their interest and enthusiasm, to show maximum diligence in the performance of their designated duties.

For plants requiring no more than limited attendance, prior experience is not overly important and the operator may be selected on the basis of certain minimum qualifications. These should include high school graduation, a reasonable degree of common sense, an ability to think clearly and a proven record of mechanical aptitude. Men thus endowed have no difficulty in learning the fundamentals of sewage treatment processes and generally become industrious, efficient and orderly operators.

2. Furnish the services of an engineer, or at least of an experienced operator, for a period sufficient to indoctrinate the regular attendants and to familiarize them with the nature and scope of their duties. This initial period should generally be relatively short, particularly in installations large enough to require the employment of an experienced superintendent. In smaller plants where only part-time and possibly inexperienced operators are to be used, the training chore may be more difficult and should be planned accordingly. Plant functions have to be fully explained, settings of automatic control devices must be adjusted, maintenance procedures established and essential laboratory tests explained and demonstrated. It also is usually desirable to call in manufacturer's representatives and have them explain the function and care of their particular piece of equipment. Likewise, a representative of one of the oil companies can assist in setting up a lubrication schedule for all equipment, specifying what oil or lubricant to use and when to use it.

3. Assemble a complete file, preferably in bound covers, of all descriptive

leaflets, bulletins, and other publications pertaining to operation and maintenance of mechanical and electrical equipment. An adequate reference file of manufacturers instructions is indispensable and should be available by the time the plant is ready to operate. The design specifications should require the contractor to furnish manufacturer's instructions for each piece of equipment and the resident engineer or inspector should put the file together during plant construction.

4. Prepare typewritten or printed instructions for operation and maintenance procedures. These shoud begin by giving the dimensions, capacities and functions of the major units in the treatment process and should be followed by a sufficiently detailed description of essential operating techniques. Attention should be directed in particular to the setting and adjusting of mechanical control devices and to such procedures as grit handling, scum removal, sludge pumping, basin cleaning, equipment lubrication, digester operation and heating, and withdrawal of sludge and supernatant from digestion tanks. In addition, procedures for necessary laboratory tests should be described and a list prepared of the equipment and solutions needed for each test. The proper technique for routine tests should be demonstrated, the significance of the results explained, and the operator told what to do to correct any deviation from normal performance.

5. Design report forms for the proper recording of necessary operating data. While record requirements vary over a wide range, depending on plant type and capacity and individual inclinations, it is desirable to provide at least two forms, one for daily entries and the other for a monthly summary. The daily form normally should include spaces for such items as power meter readings, sewage flow, raw sludge pumping, digester gas production, sludge removed from digester, and results of control tests. Space should be provided also for recording various temperature readings, for describing special operations, and for listing other items of particular interest.

The monthly summary is simply a form to record by months the totals of the various operating figures entered on the daily report form. Space should be provided for totals and averages, for total hours of operation of major pieces of equipment, and for notes and comments on the recorded data.

6. Design and set up a card system for maintenance duties, using either standard index or edge-punched cards. A card record system simplifies the job of organizing and scheduling routine maintenance activities and, if properly kept, provides a valuable history of all work done in the inspection, testing, cleaning, lubrication and painting of equipment.

7. Maintain a continuing and lasting interest in plant performance either by occasional visits as time and opportunity permit or preferably through the medium of regular visits in a consulting capacity. Service on a consulting basis is a mutually beneficial arrangement, providing the client with helpful counsel and guidance, and the engineer with useful information for future designs. Under such a program visits should be made at a frequency of not less than once every three months and should be devoted primarily to a review of operating records and a discussion of current problems. At small plants it may be desirable to run field tests and to collect samples for laboratory analysis. Each visit should be followed by a letter report commenting on the recorded operating data, on the performance and appearance of the plant, on needed improvements and on results

of the field and laboratory determinations.

A design engineer who undertakes to serve as consultant on operation must be something more than a neophyte in the field of sewage treatment. He should be a man who has been tempered by the fires of actual operating experience and capable of finding an answer to the occasionally complex problems that may develop in even the best designed installations. Moreover, the authors are of the opinion that this type of service can best be obtained from a qualified consultant and that it should not be regarded as a function of the state authorities. Granted that a limited degree of state supervision is inevitable, perhaps even desirable, it is nonetheless axiomatic that a city engaged in the operation of a sewage treatment plant should know more about its own problems than any regulatory agency. Without that knowledge and without a willingness and ability to handle problems at a local level, the door is open for state paternalism and a further encroachment of bureaucratic control.

Reimbursement for Operational Assistance

As previously noted the schedule of fees recommended by the American Society of Civil Engineers makes no provision for services beyond completion of construction. How, then, is an engineer to be compensated for any assistance he is expected to furnish in starting a new plant? Some engineers, recognizing that help is bound to be needed and that it is hopeless in most cases even to suggest additional compensation, merely count it as part of their professional service. These engineers, realizing the mutual benefits which stem from getting a plant off to a good start, undertake the additional jobs of training plant operators, preparing written instructions, providing printed forms for operation and maintenance records, and visiting the plant at regular intervals for a suitable period.

Since the percentage schedule is based on the assumption that each design job should earn no more than a reasonable fee, it is manifestly unfair to an engineer to saddle him with the burden of free assistance on matters of operation. Two remedies appear to be available. The first is the relatively simple expedient of walking off the job and getting the city either to employ a consultant or to call upon the state authorities to furnish the required services. In the latter event, the cost would be borne not by the benefitted community, but by the tax-paying public as a whole. The second and much to be preferred remedy is that of amending the free schedule to cover initial operation. Even better perhaps, would be a section recommending that services be provided on either a lump sum or per diem basis. When thus fortified, the designer will no longer be called upon to donate his time, operational help will be available on a reasonable mutually acceptable basis and initial problems will be disposed of more promptly.

Your help in planning the 1956 meeting is wanted. See page 1244.

FUNGI FROM TRICKLING FILTERS

By Albert E. Feldman

*Assistant Biologist, Massachusetts Department of Public Health, Lawrence Experiment Station, Lawrence, Mass.**

The knowledge of the use of trickling filters in treating substances in sewage is well established. However, information on the different kinds of organisms living in the filter slime is not well known. It would be extremely useful to learn how the various flora and fauna perform their functions. Before an investigation can be made on the functional properties of the trickling filter organisms, it is first necessary to isolate and identify them. Since the different kinds of organisms in the filters are too numerous to study at one time, it was necessary to limit the investigation to one group. The fungi were chosen for initial study.

This paper reports on six species of fungi isolated from three trickling filters receiving a dilute chemical waste of high organic content. The filters were of the high-rate type, 6-ft. in depth and 6-in. in diameter, filled with ¾-in. stone. Following are the loading characteristics of the filters:

Filter number	861	863	865
Flow rate (m.g.a.d.)	2	1.5	3
Recirculation ratio	5:1	5:1	4:1
B.O.D. load, 5-day (lb./acre ft.)	5,300	4,350	1,484
Hydraulic load (m.g.a.d.)	12	9	15

Experimental Methods

Slime was removed from the stones in the upper surface layer of the trickling filter and was thoroughly mixed. Portions were streaked on Czapek's agar in 9-cm. petri dishes. The inoculated agar was incubated at room temperature on a laboratory bench well illuminated by sunlight. Usually, the growths on the plates were distinctive enough to be seen and easily subcultured in two days, although certain species required as long as a week before full development. Subcultures were made using Czapek's agar until pure cultures were attained.

Results

The molds isolated were the more prominent ones and the results should not be considered as conclusive. The species isolated were as follows:

Genus and Species		Source (Filter No.)
Fusarium	*solani*	865
Fusarium	*roseum*	863
Fusarium	*moniliforme*	865
Geotrichum	*candidum*	861
Geotrichum	*sp.*	865
Alternaria	*tenuis*	865

Discussion

Geotrichum candidum is a common mold on trickling filters and has been observed by various workers whose researches are reported by Cooke (1). Various species of *Fusarium* are associated with polluted water and sewage (1). Species of *Alternaria* are associated with air (2) and soil (3). In addition to the six molds isolated, large numbers of the bacterium *Zooglea ramigera* were found in filter number 865.

Summary

Six species of fungi and one predominant species of bacteria were ob-

* Presently, Chemist, New York City Department of Water Supply, Gas and Electricity, Catskill Laboratory, Ashokan, N. Y.

served living in high-rate trickling filters receiving a diluted chemical waste of high organic content.

Acknowledgment

The author wishes to thank Wm. Bridge Cooke, Mycologist, of the Robert A. Taft Sanitary Engineering Center, Cincinnati, Ohio, for his identification of the six molds.

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CALIFORNIA HERE WE COME!

Following a record attendance at the October, 1955 Annual Meeting of the Federation in Atlantic City plans are already taking shape for the 1956 Meeting in Los Angeles, Calif., October 8–11, 1956. Chairman of the Local Arrangements Committee, Mr. Arthur Pickett, has the subcommittee chairmen busy planning a golden welcome for Federation members and guests.

In this day of fast strains and jet planes, California is quite close in time to the Federation membership in the Western Hemisphere. Because of the outstanding sights available to travelers in the Western States, many members will include the "L.A." meeting in a vacation trip.

To help in convention planning, we would appreciate receiving information from those who hope to attend the Los Angeles Meeting. Will you please send a short letter or post card to the office of the Federation Secretary stating your interest (or lack of it) in the following possibilities:

1. A special convention train making up in New York City and connecting at Chicago for a scenic trip West with side excursions—
   (a) enroute to Los Angeles, or
   (b) returning from Los Angeles
2. Post-convention trip to Hawaii by steamship or air.

Your response by January 15, 1956, will be greatly appreciated.

PARTICLE-SIZE DISTRIBUTION OF GROUND GARBAGE

By E. Robert Baumann, Darrell A. Dorman, and Marvin T. Skodje

Respectively, Associate Professor, Research Assistant † and Associate in Civil Engineering, Engineering Experiment Station, Iowa State College, Ames, Iowa

The establishment of a uniform particle-size distribution code for garbage ground in household garbage grinders and a reproducible procedure to determine the distribution have been recommended by Wolff (1). The need for uniformity is evident from the fact that some city codes are not in agreement with particle-size distribution test results obtained with typical ground garbage. At present no standardized procedure for determining the particle size of ground garbage exists. Until such a procedure is established, codes will be meaningless. A proposed code and procedure are presented by the authors.

In conducting studies of the heat treatment of garbage, it was necessary to develop a reproducible procedure for determining the particle size of ground garbage and to obtain representative data of ground garbage, particle-size distribution.

Numerous experimental procedures were tested in the laboratory and modified repeatedly in developing a satisfactory test procedure (2). These preliminary tests indicated that a satisfactory procedure would involve:

1. The suspension of the ground garbage in a volume of water sufficient to facilitate the passage of particles through a set of standard sieves.
2. The passage of the ground garbage suspensions separately through each sieve of a set of sieves to facilitate handling and sieving.
3. The use of a synthetic detergent to cut the grease in the garbage. The grease otherwise tends to plug the screen openings in the sieve and agglomerate the particles of ground garbage.

With these factors determined, the two-container method was developed for determining the particle-size distribution of ground garbage.

Data Analysis

The percentage of solids passing each sieve is calculated on the basis of total weight of dry solids. The total garbage solids recovered in the test may be determined as follows:

$$\begin{array}{c}\text{Total weight}\\ \text{of solids}\\ \text{recovered}\end{array} = \begin{array}{c}\text{weight of}\\ \text{solids}\\ \text{retained on}\\ \text{sieves}\end{array} + \begin{array}{c}\text{weight of}\\ \text{total solids}\\ \text{in dilution and}\\ \text{rinse water}\end{array} - \begin{array}{c}\text{weight of}\\ \text{added solids,}\\ \text{tap water}\\ \text{and detergent}\end{array}$$

† Presently, Engineer, City of Milwaukee, Milwaukee, Wis.

A check on the accuracy of the test can be made by comparing the weight of the solids recovered in the test with the weight of the garbage solids added to the top sieve. In a satisfactory test, at least 95 to 97 per cent of the solids added to the top sieve will be recovered on the sieves or in the dilution and rinse waters.

The data should be expressed graphically by plotting the average of two or more runs on logarithmic paper with the per cent passing plotted as the ordinate and sieve opening in millimeters as the abscissa. This plot will normally approach a straight line. The results of two runs may normally be expected to agree with a maximum variation of 3 to 5 per cent.

TABLE I.—Particle-Size Distribution of Ground Garbages

Trial	Run	Sieve Size or Number[1]							
		⅜"	3/16"	¼"	⅛"	#10	#20	#40	#60
No. 9	1	100	91.6	82.6	78.8	74.5	67.2	61.6	60.8
	2	100	91.2	82.4	79.2	75.7	67.6	62.4	61.6
	Avg.	100	91.4	82.5	79.0	75.1	67.4	62.0	61.2
No. 10	1	100	95.6	85.8	81.2	74.6	68.4	61.3	59.5
	2		95.4	85.2	81.9	75.2	69.9	62.7	60.9
	3		95.7	86.1	81.8	75.1	66.9	60.5	59.1
	Avg.		95.6	85.7	81.6	75.0	68.4	61.5	59.8
No. 11	1		96.9	88.4	85.6	82.6	79.4	74.6	73.2
	2		96.9	89.2	86.4	83.7	78.9	75.4	74.2
	3		96.5	85.6	82.0	78.7	73.5	69.7	68.3
	Avg.		96.8	87.7	84.7	81.7	77.3	73.2	71.9
No. 13	1		86.0	76.7	73.0	68.3	59.0	54.3	53.5
	2		90.3	81.0	77.6	73.1	61.4	56.8	55.9
	Avg.		88.2	78.8	75.3	70.7	60.2	55.6	54.7
Detergent effect	0 g.		97.0	91.0	87.0	82.2	63.1	54.5	53.4
	10 g.		97.8	91.7	88.8	83.6	63.8	56.2	55.5
	20 g.		96.3	91.9	89.1	84.4	63.8	59.1	58.3
	30 g.		97.7	92.2	88.7	83.9	64.2	57.3	56.6
	Avg.		97.2	91.7	88.4	83.5	63.7	56.8	56.0
Reproducibility test	1		83.2	72.1	69.2	65.9	61.6	52.3	50.0
	2		86.2	73.8	70.9	67.5	63.2	54.0	51.5
	3		85.3	75.3	70.3	67.3	62.9	55.7	52.9
	4		85.1	73.1	69.7	66.7	62.8	52.7	49.4
	Avg.		85.0	73.1	70.0	66.9	62.6	53.7	51.0
Effect of grinding head size	3/4"		73.2	63.2	60.5	57.0	52.5	48.9	48.0
	1/2"		94.9	77.5	72.5	67.8	62.2	58.4	57.0
	3/8"		99.7	84.1	77.7	72.8	68.1	63.4	62.4
	3/16"		100.0	99.0	86.9	77.7	68.2	65.1	63.2
Domestic garbage grinder	1		99.2	91.8	89.1	85.2	76.1	66.1	64.2
	2		99.0	95.8	92.6	82.9	70.8	65.4	60.8
	Avg.		99.1	93.8	90.8	84.0	73.4	65.8	62.5
Average of data using 3/4-in. head			92.4	83.3	79.8	75.5	66.6	60.5	59.1

[1] All values are per cent passing.

Verification of the Proposed Method

The reproducibility of the two-container method of determining the particle-size distribution of ground garbage was verified in two ways:

1. The particle-size distributions of numerous ground garbages were determined in two to five identical runs by the same person.

2. The particle-size distribution of a ground garbage was determined in identical runs conducted by different people.

The data for the reproducibility test listed in Table I and plotted in Figure 1 were obtained by making four identical runs with aliquot portions of the same ground garbage. The maximum variation in this test from the mean value of percentage passing a given sieve size was two per cent (Table II). The maximum variation between any two values was only 3.4 per cent. The variations between runs made in trials 9, 10, 11 and 13 are also tabulated. Considering the maximum variation from the mean value as a measure of the reproducibility of the test procedure, the data indicate an average variation in test results of 2 to 4 per cent. Because many factors affect the accuracy of the test and the desired application of the test results, it may be concluded that the procedure provides a reasonably accurate method of determining the particle size of ground garbage.

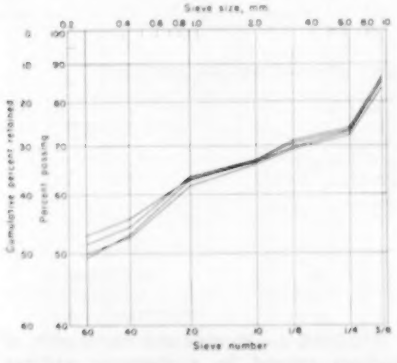

FIGURE 1.—Variation in particle-size distribution based on four runs of the same ground garbage.

TABLE II.—Range of per cent variation of results for runs using the same garbage sample[1]

Trial[2]	Number of Runs Per Trial	Range of Percentage Variation		
		From Mean	Maximum	Minimum
No. 9	2	0.9	1.8	0.2
No. 10	3	1.6	3.0	0.3
No. 11	3	3.8	5.9	0.4
No. 13	2	2.4	4.8	2.2
Detergent effect	4	2.6	4.9	1.1
Reproducibility test	4	2.0	3.4	0.1

[1] A different garbage was used in each trial.

One major factor affecting the accuracy of the test and the labor required in making the sieve analysis was the grease contained in nearly all the garbages. The grease had a tendency to agglomerate the ground particles of garbage, making it difficult or impossible to separate the garbage particles according to size. Tests were conducted, therefore, using a synthetic detergent in an attempt to reduce this problem. The detergent had to be added to neutralize the agglomerating effect of the grease without producing a soap-film barrier in the sieve openings. The amount of detergent required varies with each garbage, but can be easily determined by trial and error. The formation of suds which remain throughout the sieving was an indication that too much detergent was used.

The use of a detergent in varying amounts in the detergent-effect trial did not affect the reproducibility of the test procedure. (Tables I and II). It did, however, considerably reduce the time required for sieving the garbage when it was used in optimum amounts. If residual suds were present during sieving, the accuracy of the test was affected adversely.

Grinding

In the study of the heat treatment of garbage, a large number of tests were made using ground garbage. The garbage used in most of these tests was commercial garbage, or waste food collected from establishments engaged in mass feeding, such as restaurants, hospitals, food markets and institutional food services. The garbage was similar to that which would be disposed of through domestic garbage grinders. For that reason, the garbage contained a minimum of glass, paper and other materials which normally would not be fed to a domestic garbage grinder.

A 1.5 h.p. commercial meat chopper with a capacity of 20 to 25 lb. of beef per minute was used to grind the garbage. The chopper was equipped with interchangeable cutting plates or heads of different sizes which made it possible to control the maximum particle size of the ground garbage. Grinding head sizes used in this study were ¾-in., ½-in., ⅜-in. and 5/16-in. The diameter of the individual openings in the grinding head is used to designate the fineness of the grinding. The ¾-in. grinding head was used for all numbered trial determinations of particle-size distribution.

In the grinding trials, about 200 lb. of garbage was ground at a time. The garbage was sorted prior to grinding and stray pieces of glass and bones which would clog the food chopper were removed. The ground garbage was collected and thoroughly mixed in 20-gal. cans. The two-container method was used to determine the particle-size distribution of representative samples of the ground garbage. The data were plotted on logarithmic paper in an attempt to obtain a straight line relationship between the screen opening and the cumulative percentage passing the screen.

The data are designated by trial numbers or descriptive names. A different garbage was used in each trial. Each trial was subdivided into two or more runs from which independent particle-size determinations were obtained for the same garbage.

Test Results

More than 300 tests were made to determine the particle-size distribution of ground garbage. Typical results of tests using 8 different samples of commercial garbage are included in Table I. Trial Nos. 9, 10, 11 and 13 were routine tests made to determine the particle size of garbage ground using the ¾-in. grinder head. The trial labeled "detergent effect" was made to determine the effect of using various amounts of detergent to cut the grease in the ground garbage. The trial marked "reproducibility test" was made to determine whether similar particle-size results could be obtained with the same garbage.

Figure 2 shows the average particle-size distribution of garbage ground in the meat chopper using the ¾-in. head. The curve represents the average of the first six trials listed in Table I. The particle-size distribution varied from trial to trial because of the variation in the composition of the commercial garbage. In some cases, the

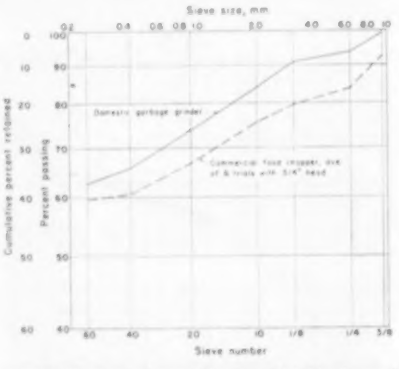

FIGURE 2.—Particle-size distribution of garbage ground in a domestic garbage grinder and in a commercial meat chopper with a ¾-in. head.

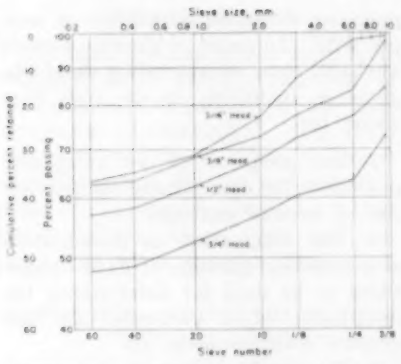

FIGURE 3.—Effect of grinding-head size on particle-size distribution of garbage ground in a commercial food chopper.

garbage consisted of leftovers from an evening meal. In other cases, the garbage consisted of breakfast foods. For this reason, the average percentage of material passing the No. 60 sieve was found to vary from a low of 51 per cent to a high of 71.9 per cent. The latter garbage consisted chiefly of breakfast wastes containing a large proportion of oatmeal wastes. If this trial was not considered, the variation would be from 51 to 61.2 per cent. In other words, with a normal garbage ground using the ¾-in. head, from 50 to 60 per cent of the material on a dry weight basis will pass the No. 60 sieve as soluble or suspended material. The material passing the No. 60 sieve, based on the average of several typical garbage samples, is composed of about 30.5 per cent soluble solids and 69.5 per cent suspended solids.

In one test the garbage was ground in a domestic garbage grinder and the results indicated that the garbage was ground finer than in the food chopper. More than 60 per cent of the ground garbage passed through the No. 60 sieve.

In another test aliquot portions of the same garbage were ground using different heads in the food chopper. Increasing the fineness of grinding increased the percentage of garbage passing each sieve in the set (Figure 3). The increment of increase in the percentage passing the smaller size sieves decreased as the grinding head size decreased.

Particle-Size Distribution Codes

The particle-size distributions required for ground garbage by various cities is shown in Table III. The city codes for Detroit, Mich. and Philadelphia, Pa. require, respectively, that 100 per cent and 95 per cent of the garbage ground by domestic disposal units be retained on a No. 40 sieve. In addition, they require that not more than 5 per cent, and 10 per cent, respectively, be retained on a ¼-in. sieve. Los Angeles, Calif., the District of Columbia, and the U. S. Govern-

TABLE III.—Particle-Size Distribution Specified by Various Codes

Code	Per Cent Permitted on Given Sieve					
	½-in. (12.70 mm.)	⅜-in. (9.52 mm.)	No. 3 (6.35 mm.)	No. 6 (3.36 mm.)	No. 8 (2.38 mm.)	No. 40 (0.42 mm.)
	Maximum					Minimum
Detroit, Mich.	0	—	5	—	55	100
Los Angeles, Calif.	0	—	35	—	60	—
Philadelphia, Pa.	0	—	10	—	—	95
District of Columbia	—	0	25	—	—	—
U. S. Government	—	—	35	60	—	—
Recommended	0	10	15	25	—	25

ment codes require that not more than 25 to 35 per cent be retained on a ¼-in. sieve. The Los Angeles code requires also that not more than 60 per cent be retained on a No. 8 sieve and the U. S. Government requires that not more than 60 per cent be retained on a No. 6 sieve.

Feasibility of Existing Codes

On the basis of the data presented it would appear that the municipal codes for Detroit and Philadelphia cannot be complied with fully. These codes require that 5 per cent or less of the ground garbage be permitted to pass through a No. 40 sieve. In these tests, an average of 60.5 per cent of the garbage ground through the ¾-in. head with the commercial meat chopper and an average of 65.8 per cent of the garbage ground through a domestic grinder passed through the No. 40 sieve. Thus, none of the tests reported would comply with this requirement. A comparison of the codes for Los Angeles, the District of Columbia and the U. S. Government with the data indicates that these codes also do not approximate results which are obtained in the field.

Recommended Code

Existing codes specifying the particle-size distribution of garbage ground in household food-waste grinders are based on the following objectives:

1. To control the maximum size of the garbage particles so no garbage will settle out in the sewers before it reaches the treatment plant.
2. To control the minimum size of the garbage particles so most of the garbage will be removed in the sedimentation process and will not contribute excessively to the secondary treatment processes.

Test data, however, indicate that some of the codes designed for these purposes would be impossible to comply with. In order to provide a more workable code, consideration should be given to including in the code provision for:

1. The adoption of an acceptable procedure for determining the particle size of ground garbage.
2. The designation or formulation of a synthetic garbage of fixed composition to be used for determining the compliance of a particular garbage grinder with the code.
3. The establishment of an acceptable range of per cent passing for standard sieves of various sizes.

The following items are recommended, therefore, for use in specifying the particle-size distribution of garbage from domestic food-waste grinders:

1. A standard method, such as the proposed two-container method, shall be used to determine the particle size of ground garbage.
2. The determination of the particle-size distribution of garbage from a domestic food-waste disposer shall be made with a garbage of the following composition:

"The garbage used to determine compliance of a domestic grinder with the code shall be a commercial garbage consisting of food wastes from an evening meal at a restaurant serving not less than 600 people."

3. An acceptable food-waste grinder shall produce a particle-size distribution for the ground garbage that falls within the following ranges:

Sieve No.	Range of Percentage Passing
½-in.	100
⅜-in.	90–100
¼-in.	85–100
⅛-in.	75–95
No. 10	70–90
No. 20	62–82
No. 40	55–75
No. 60	50–70

Acknowledgment

The data reported in this paper were collected in a study "Survival of Swine Disease Organisms in the Heat Treatment of Garbage." This study was conducted by the Iowa Engineering Experiment Station and the Division of Veterinary Medicine, Iowa State College, Ames, Iowa and was supported by a research grant from the National Institutes of Health, Public Health Service.

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APPENDIX—PROCEDURE FOR DETERMINING THE PARTICLE-SIZE DISTRIBUTION OF GROUND GARBAGE

The apparatus, procedure, and data analysis used in the Iowa State College Sanitary Engineering Laboratory for determining the particle-size distribution of ground garbage are as follows:

Apparatus

1. Set of U. S. Standard Sieves—½-in., ⅜-in., ¼-in., ⅛-in., No. 10, No. 20, No. 40 and No. 60.
2. Two galvanized cylindrical containers of approximately 10-l. capacity each, of sufficient diameter that a sieve might be manipulated partially submerged within the container during sieving.
3. One 25-l. container for collecting final wash and rinse solution.
4. Hose and shower spray head (2½-in. diameter) containing about sixty 1/32-in. diameter holes.
5. Thirty tared 125-ml. evaporating dishes.
6. Two tared 600-ml. beakers.
7. Balances.
8. Drying oven (preferably a vacuum or inert gas oven to prevent oxidation) capable of maintaining a temperature of 100° C. for drying mosture samples.
9. 100-ml. pipette.
10. Spatula.
11. Impervious work area, about 2-ft. by 2-ft., for collecting garbage from sieves.
12. Watch glasses to cover 600-ml. beakers.

Two-Container Procedure

1. Mix thoroughly the garbage to be tested.
2. Select three 100-ml. representative samples for a moisture determination, place in three tared evaporating dishes and weigh on an analytical balance immediately after each sample is taken. Speed is essential since garbage loses moisture rapidly. Place the three evaporating dishes used for the moisture determination in a 100° C. oven and dry to constant weight, approximately 48 hr.
3. Select, simultaneously, two 600-ml. representative samples of the garbage for the sieve analysis, place in two tared beakers and weigh immediately. Determine the weight to the nearest gram.
4. Wash the contents of one of the 600-ml. beakers into one of the 10-l. containers with the aid of the shower spray.
5. Add a predetermined weight of detergent, if necessary, to cut the grease and facilitate the sieving action. The amount of detergent added should not cause the formation of residual suds.
6. Dilute and suspend the garbage sample in the 10-l. container with the spray, filling the container about ⅔ full of water. The rate of spray discharge should be about 10-l. per min.
7. Pour the contents of the 10-l. container through the largest size sieve,

collecting the filtrate in the second 10-l. container. Wash particles of garbage out of the container with the spray.

8. With the sieve submerged one-half in the liquid of the second container, sieve the garbage. At no time should the top lip of the sieve be submerged nor should the washing action cause splashing.

9. Rinse the sieve contents with the spray after it has been determined visually that the particles remaining on the sieve will continue to be retained. The spray discharge rate should be 2- to 5-l. per minute.

10. Empty the contents of the sieve on the collection area.

11. Place the particles in a tared evaporating dish and evaporate in a 100° C. oven to constant weight, approximately 48 hr.

12. Repeat steps (5) through (11), using the next smaller sieve each time until all of the sieves in the set have been used.

13. Pour the wash and rinse water remaining in the 10-l. containers into the 25-l. container and weigh to the nearest gram.

14. Agitate the solution; remove three representative 100-ml. samples and place in three separate tared evaporating dishes. Agitate the solution before each of the three samples is removed. Evaporate the samples to dryness on a steam bath and dry to constant weight in a 100° C. oven, approximately 24 hr.

15. Empty and rinse the 25-l. container and weigh to the nearest gram.

16. Repeat steps (4) through (15), using the second 600-ml. sample of garbage.

17. Remove the dried samples from the oven, cool in a desiccator and weigh on an analytical balance. Speed is essential when weighing, since the dried samples are exceedingly hydroscopic.

18. Determine the moisture content of the detergent and the total solids in the water used in the test.

Modifications for Testing Domestic Garbage Grinders

If the two-container method is used to determine the particle size of garbage ground through a domestic grinder, the following steps should be substituted in the procedure for steps 4, 5, and 6.

4. Wash the unground garbage from one of the 600-ml. beakers into the domestic disposal unit to be tested and operate the grinder.

5. Collect the diluted and suspended ground garbage from the disposal drain outlet in the 25-l. container.

6. Add a predetermined weight of detergent, if necessary, to cut the grease and facilitate the sieving action. Agitate slowly. The amount of detergent added should not cause residual suds.

Industrial Wastes

TREATMENT OF ACID WASTES WITH CALCITE*

By Vincent de P. Lukas

Sanitary Engineer, General Electric Company, Schenectady, New York

The Clock and Timer Department of General Electric Company manufactures "Telechron" electric clocks and small self-starting, synchronous motors for meters and instruments. In 1947 modifications to an old street car barn resulted in the present manufacturing facilities at Worcester, Mass. With new machinery and equipment installed, component production and assembly was started on an expanded scale.

At present a portion of the manufacturing facilities are devoted to defense items which require close tolerances and volume production machinery. Acid wastes from the plating of many of the intricate parts for defense items and synchronous motors constitute the principle industrial waste.

After deciding to expand operations, industrial waste treatment was given consideration in the planning for the new operation even though all wastes were to be discharged to the Worcester sewer system. This action was prompted because acid wastes from the many wire and metal companies in Worcester were damaging the sewers which, compared to those in use in most systems, were designed to carry slightly alkaline sewage. In 1952, the city adopted an ordinance to control industrial wastes.

The Treatment Unit

Design Data

The acid waste treatment system of upflow limestone beds was designed from the basic data obtained from studies by Gehm (1) (2) and Reidl (3). The system treats all the waste water from the washing, plating and rinsing operations. These operations were expected to produce approximately 14,000 gal. per hour of waste with an acid content of 379 p.p.m. as sulfuric acid. It was anticipated that the expanded plant would use approximately 20,000 gal. of various acids and 55,000,000 gal. of water per year in the plating room operations.

The acid waste treatment system was built for about $17,000 and was placed in operation in July of 1947. The most unique feature of this treatment system is its location. Due to the lack of available land the system was constructed below the floor in the plating room (Figure 1). All wastes from the plating room operations spill onto the subfloor below the wooden floor and flow by gravity into concrete channels located down the center and along the edges of the subfloor. The wastes flow through two ¼-in. mesh stainless steel screens in series, which were installed after the treatment system was built. A plan of the treatment system is shown in Figure 2.

Tanks

The wastes flow through a skimming tank into a sludge tank and then into

* Presented at 5th Joint Meeting, New England Sewage and Industrial Wastes Assn. and New York Sewage and Industrial Wastes Assn.; Albany, New York; June 13, 1955.

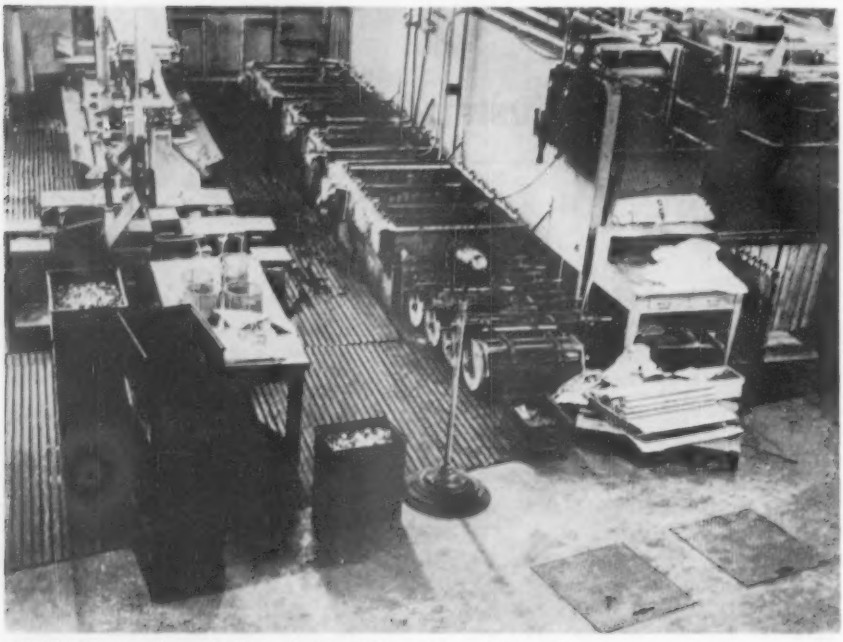

FIGURE 1.—General view of the plating room. The acid waste treatment system is located under the floor (note manholes).

a settling tank. The combined capacities of the tanks at minimum and maximum liquid levels are 8,000 and 15,600 gal. respectively, resulting in a storage capacity of 7,600 gal. or more than one-half hour detention based on the design flow rate of 233 g.p.m. At the present average flow of 315 g.p.m. the detention time is almost 24 minutes.

These tanks may be considered as one large concrete dilution tank that receives all the wastes from the plating operations. The first section acts as

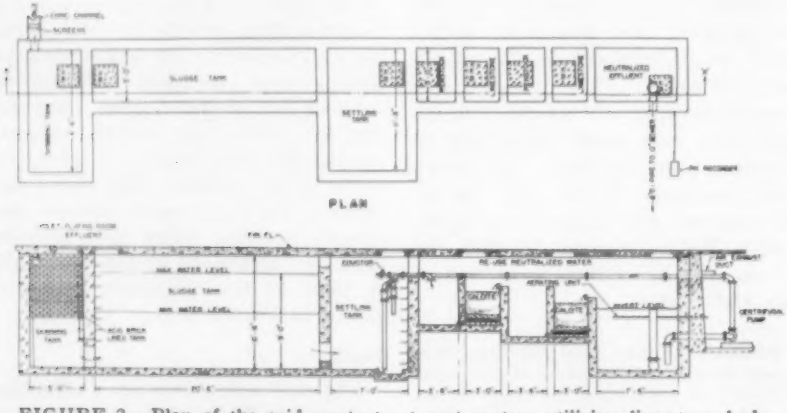

FIGURE 2.—Plan of the acid waste treatment system utilizing limestone beds.

an oil and grease separator, in addition to retaining floating solids. In the second section the heavier solids settle, while the third section contains fairly clear liquid. The size of this tank provides only limited equalization, particularly when concentrated acid and alkaline solutions are dumped.

Eductor

From the final settling tank the wastes are syphoned into a penstock by means of an eductor, designed to handle the waste at rates of 233 g.p.m. or greater. This is accomplished by pumping neutralized effluent through a nozzle to the pressure side of the eductor. The flow is increased greatly with a drop in pressure thereby inducing the syphon action. The treated wastes mix with the untreated wastes and flow into the first penstock.

Limestone Beds

Two limestone beds each 5 ft. by 3 ft. with a 2-ft. depth are provided in series. From penstock or compartment No. 1 the waste flows up through the first limestone bed at an average rate of 26 g.p.m., per square foot, into penstock or compartment No. 2 and then up through the second limestone bed into the final tank.

Final Tank

In this final tank is located the suction line with a recirculating pump which can handle up to 700 g.p.m., returning 233 g.p.m. to the aeration nozzles located in the last tank. The turbulence liberates some carbon dioxide and the pH of the effluent is raised slightly The treated wastes are discharged through an 8-in. pipe to the city sanitary sewer.

Gases and fumes are removed by vent ducts connected to a mechanical ventilation system. The tanks are provided with sufficient manholes to permit easy access for cleaning, inspection and maintenance. Acid proofing was accomplished by using acidproof brick lining in the skimming tank and asphalting all other interior surfaces.

In the event of a shutdown of the centrifugal pump the system can operate by gravity. The baffle wall between the first compartment and the limestone bed is high enough to permit this action. A pH recorder was provided as a check on acidity of the effluent.

Operation

The original limestone used was "size No. 2 calcite crystals" with crystals 2 to 3 mm. in diameter. This calcite gave considerable trouble during initial operations due to a build up of fine, thightly packed insoluble solids which reduced flow through the beds and gave a false indication of the effective depth of the limestone. This calcite was replaced with another product known as "Roll-Cal H," a high grade poultry limestone. The new source of calcite analyzed 97.2 per cent as calcium carbonate as compared to

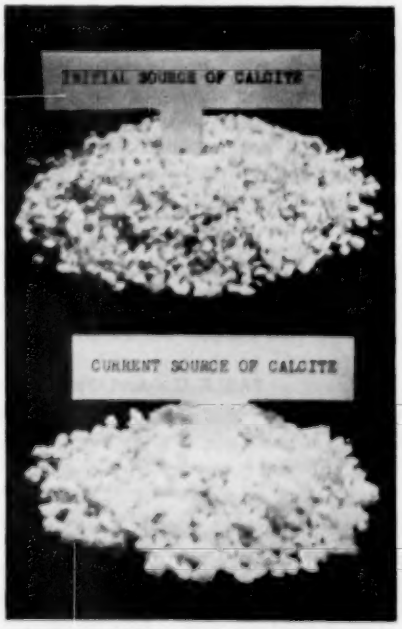

FIGURE 3.—Comparison of initial and current source of calcite.

TABLE I.—Acid Waste Treatment System—pH Readings

Time (A.M.)	Sample	pH
9:00	Inlet	2.91
9:05	Skimming Tank	3.15
9:10	Compartment No. 1	4.5
9:15	Compartment No. 2	5.5
9:20	Outfall	5.7

83.5 per cent for the original calcite. The two calcites are shown in Figure 3.

The Roll-Cal cost averaged about $17.80 per ton, but last fall it was increased to $25 per ton. For the past two years daily records of the additions to the two neutralizing beds show that bed No. 1 required an average of 2,500 lb. per month and bed No. 2 about 1,100 lb. per month. Calcite monthly requirements vary with production.

The use of two limestone beds in series has proven to be sound practice and insures better treatment of the acid wastes. If, for some reason, the first bed should have insufficient height of calcite due to excessive use, or if channeling should take place, any residual acidity would be partially neutralized in the second bed. Also by having double the amount of limestone area available, the need for frequent additions of calcite is reduced.

Analytical Data

Detailed analytical data indicate the influent pH of the wastes to be between 2.1 and 2.5. The pH averages 2.4 and is raised to about 3.5 following the first limestone bed. After up-flow through the second limestone bed the pH is raised to about 4.7 (4.5–5.2). The treated wastes often exceed a pH of 5. Aeration in the last tank increases the final pH only slightly—about 0.05 at the maximum.

Typical pH values at various points in the acid waste treatment system are given in Table I.

In May 1951 the Massachusetts Department of Public Health made a survey of the acid waste treatment system and composited samples at six points, with portions of samples collected every 30 minutes. The results of the survey are given in Table II.

The system was designed to recirculate 700 g.p.m. of the treated waste to give a dilution factor of 2:1. An increase in acid waste flow to 315 g.p.m. plus the recirculated flow of 467 g.p.m. to the first compartment preceding the calcite beds caused flooding of the system. Therefore, it was necessary to reduce the recirculation to 167 g.p.m. (plus 233 g.p.m. for aeration). The efficiency of the system remained the same, which indicated that dilution was not contributing materially to the treatment of the acid waste.

A two-week trial period was selected in April, 1955 to determine if recirculation should be discontinued. Twice daily, samples were collected for pH and acidity determinations on the influent and effluent. The first week the

TABLE II.—Acid Waste Treatment System—Chemical Analyses

Test	Sampling Point					
	Influent	Eductor	Compartment No. 1	1st Overflow	2nd Overflow	Effluent
Suspended solids (p.p.m.)	11	13	10	6	10	12
Chlorides (p.p.m.)	35	58	58	51	58	58
Acidity (p.p.m.)	50	50	14	10	10	10
pH	3.0	3.0	3.7	4.7	5.0	5.0
Iron (p.p.m.)	2.0	3.0	2.8	2.6	2.5	2.8
B.O.D., 5-day (p.p.m.)	1.4	1.0	0.7	1.1	0.9	1.3

recirculating pump was in operation while during the second week the system was operated without recirculation. Both weeks were normal production periods with similar operations being carried on in the plating room. Equal efficiencies were obtained with and without recirculation. Additional studies will be made before discontinuing the use of recirculation.

Maintenance

Minor maintenance of the acid waste treatment system is the responsibility of the plating room personnel. The depth of the calcite beds is checked twice daily, usually one hour after the start of operations in the morning and in the early afternoon. Calcite is added as required, but never less than in bag lots of 80 or 100 lb.

The stainless steel screens are cleaned three to five times per day. Since the installation of these screens, the skimming tank requires cleaning on the average of once every two weeks instead of every three to five days as was the previous practice. Scum and other floating material is skimmed off at the manhole and placed in covered metal cans and disposed at the city dump.

The pH of the effluent is checked periodically on a pH meter and is compared with the reading on the automatic pH recorder. Frequent checks are made of the chart on the pH recorder by the operator. The time of dumping of any solutions also is recorded on the chart.

All the calcite is removed from the beds and tanks every three months. With the original acid-proof linings frequent repairs were necessary to the tanks containing the limestone beds. The repairs consisted of filling holes with roof cement and finishing with a coating of "No-Tar" roof coating. The average cost was about $48 per cleaning. A fresh supply of calcite was then added to bring the depth of the beds to two feet.

Problems and Solutions

In 1952 the calcite tanks were relined with acid resistant red quarry tile and acidproof cement from the bottom to one foot above the waste water level at a cost of $360. The use of tile has proven to be successful and eliminates maintenance due to corrosive action of the waste in the calcite tanks, a costly maintenance item.

During vacation shut down each year, all the tanks are drained and the walls and piping are checked and repairs made when necessary.

Originally, the subfloor in the plating room required recovering with four plys of felt and hot tar every two years at a cost of about $400. Recently, asphalt coated jute was used in place of the roofing felt at a small increase in cost. If replacement every four years is realized, the saving will amount to $150 annually.

Yearly maintenance of the concrete channels requires patching corroded concrete and asphalt relining at an approximate cost of $125. It is estimated that the proposed use of jute and asphalt will save $60 per year or about one-half of current maintenance cost.

The original ductwork venting the acid waste treatment system was fabricated from copper bearing steel and coated inside and out with an asphalt paint. These ducts corroded out in six months and were replaced with stainless steel that rusted and pitted in seven months. Finally, the ducts were constructed of marine plywood and coated inside and out with asphalt paint. To date, a savings of about $1,000 has been realized by the use of plywood ducts in place of the others. A recent check showed that the plywood is holding up well and should last many more years.

The stainless steel blower used for forced ventilation of the tanks corroded and required replacement at a cost of $1,000. The present rubber covered blower and housing have been

in use for five years and are giving satisfactory service. The initial cost of the stainless steel blower was $600 and it lasted only 14 months.

Operating Costs

Daily maintenance averages two hours of labor and costs approximately $900 per year. Cleaning of the calcite beds, exclusive of the cost of calcite replacement, amounts to $100 annually. Yearly maintenance is between $500 and $600, depending on the extent of repairs (about $480 for labor), bringing the total annual cost of maintaining the system to $1500.

A reasonable estimate of the cost of treating the acid waste is $0.17 per 1,000 gal. based on an annual flow of 28,000,000 gal. The average annual costs are:

Limestone	$ 490
Daily maintenance	900
Yearly maintenance	600
Power	1,500
Depreciation	1,200
Total	$4,690

Summary

1. The General Electric Company manufacturing facilities at Worcester, Mass. discharge a waste having a pH range between 4.5 and 5.2 which is satisfactory to city and state authorities. Considering future production plans this should remain true for a long time.

2. The most significant feature of the acid waste treatment system is its location below the floor in the plating room. Satisfactory operation of this system proves that when space is critical an installation of this type is feasible.

3. The application of 26 gal. per minute per square foot of a waste having an initial pH of 3 through a 2-ft. bed of limestone gives efficient operation.

4. Recirculation and aeration does not increase the efficiency of the treatment system.

5. Maintenance costs were materially reduced after operating experience was obtained.

6. The cost of treating the acid waste was $0.17 per 1,000 gal.

7. The use of a high grade poultry limestone eliminated operational difficulties, and justified the extra cost. Industrial waste may be treated with "chicken feed," but certainly the cost isn't so.

Acknowledgment

The writer wishes to express his appreciation to A. Baia, K. M. Foote and R. Gladu, Telechron personnel, who were most helpful.

Appreciation is also expressed to K. S. Watson for his counsel and W. H. Taylor for providing the analytical data on the survey by the Massachussets Department of Public Health.

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Your help in planning the 1956 meeting is wanted. See page 1244.

TREATMENT OF ANODIZING WASTES BY ION EXCHANGE *

By Lacern M. Corcoran

Water and Waste Engineer, Aircraft Gas Turbine Division, General Electric Company, Evendale, Ohio

The Aircraft Gas Turbine Division of the General Electric Company manufactures jet engines at Evendale, Ohio for the U. S. Air Force. The Evendale facilities are served by the Cincinnati Metropolitan Sewage Disposal System and must comply with its rules and regulations.

Principle waste problem at the A.G.T. Division facilities is the treatment of chromic acid waste from the anodizing of aluminum parts. The Cincinnati Metropolitan Sewage Disposal System has set the limit for the pH of liquids discharged to the sewer system at 5.5 to 9.5 and the chromium content at 5 p.p.m. as hexavalent chromium.

The wastes consist of rinse waters having a chromium content greater than the code limit of 5 p.p.m. of hexavalent chromium, and the anodizing bath has a concentration of 7 oz. of chromic acid per gallon. The recent development of stable ion exchange resins capable of withstanding attack by the more dilute solutions of chromic acid, has provided a profitable means of purification of contaminated anodizing baths and chromium recovery from the rinse waters.

The rinse water and the spent bath can be treated by two methods:

1. Chemical reaction which reduces the hexavalent chromium to the trivalent form, then neutralizing to an acceptable pH.

2. The ion exchange method of removing the aluminum from the spent bath and the recovery of chromic acid from the rinse waters.

When the anodizing bath becomes contaminated with approximately 10 g. per liter of aluminum oxide, the high electrical resistance caused by this impurity makes it necessary to change the bath. The contaminated solution must then either be treated or wasted.

Treatment Methods

Chemical Method

The chemical method consists of treating the spent anodizing bath with ferrous sulfate, sodium meta-bisulfite, or sulfur dioxide. The use of ferrous sulfate, sodium meta-bisulfite, or sulfur dioxide requires that the reduction be carried out at a pH of 3. Additional sulfuric acid is needed to keep the pH in this range; therefore, more alkali is needed to precipitate the trivalent chrome in bringing the pH to 8.5. Sulfur dioxide appears to be the best reducing agent as the reduction goes to completion at a slightly higher pH. In addition, it is easier to handle cylinders of sulfur dioxide gas than dry chemicals.

At a pH of 3, the reaction goes to completion with a slight lowering of the pH, and will reduce the hexavalent chrome. The theoretical amount of sulfur dioxide required is 184.5 lb. per 100 lb. of hexavalent chrome present. At full production it is estimated

* Presented at 1955 Annual Meeting, Ohio Sewage and Industrial Wastes Conf.; Columbus, Ohio; June 22–24, 1955.

that the bath may be contaminated in one month, requiring treatment of 10,000 gal. of anodizing solution.

The rinse waters containing 10 to 20 p.p.m. of hexavalent chromium are treated to meet the city of Cincinnati code requirements of 5 p.p.m. as hexavalent chrome. It is estimated the cost to treat the chrome in the rinse waters will be approximately $2,000 and that approximately $2,250 worth of chromic acid will be lost. This is an approximation because the number of parts going through the bath varies from day to day.

Ion Exchange Method

A more economical solution to this problem is the ion exchange method of purification and recovery (Figure 1). The contaminated anodizing solution is passed through a cation exchanger where the metallic ions are removed and the treated solution is returned to the tank. The bath is held at a pH of 0.8 and 0.283 g. of aluminum is deposited in the bath per ampere hour. As the aluminum parts are anodized and some aluminum is deposited in the bath, the pH rises. The normal practice is to add additional quantities of chromic acid to hold the pH at 0.8. The bath must either be disposed of or recovered when the additions equal 10 per cent of the total weight of the original bath.

A small portion of the 10,000 gal. bath is treated each day to keep the pH at the operating level of 0.8. In this way the entire tank contents do not have to be discharged and only a small amount of make up solution is necessary. The resin is regenerated by a 10 per cent solution of 66° Be' sulfuric acid, which removes the aluminum and restores the resin. Thus all the chromic acid is saved to be reused at the expense of the relatively cheap sulfuric acid.

The cation exchanger or chrome purifier is 48-in. in diameter and 84-in. high, and lined with koroseal. The unit is charged with 40 cu. ft. of a styrene base resin, especially resistant to the oxidizing effect of chromic acid. The resin bed is supported on a bed of graded quartz. The transfer pump is a Duriron centrifugal pump delivering 40 g.p.m. at a 20 ft. head driven

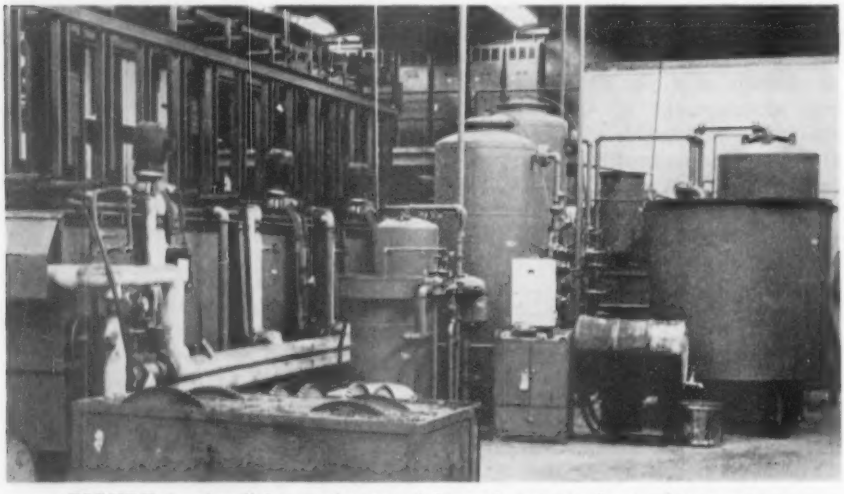

FIGURE 1.—Anodizing equipment (left) and compact ion exchange waste treatment unit (right).

by a 1 h.p. motor. The steel tank for making up the fresh 10 per cent sulfuric acid is 5-ft. in diameter and 5-ft. high lined with neoprene. The acid recovery tank has the same dimensions and lining. On the outlet from the unit there is a conductivity instrument to indicate the point when aluminum is no longer removed by the ion exchange unit.

Rinse Water System

The rinse waters flow to a 5-ft. × 5-ft. × 4-ft. steel collecting tank containing a pH cell connected to a pH indicating controller. The pH controller regulates the flow of sodium hydroxide pumped through the unit. A two to three per cent solution of sodium chromate is eluted and pumped through the chromium purifier, where it is converted to pure chromic acid to be returned to the anodizing tank.

The anion exchanger for absorbing the chromic acid is 36-in. in diameter and 96-in. high and is charged with 24 cu. ft. of a strong base anion resin supported on a graded quartz bed. The system has sampling cocks and rotameter to show flow rates. The regenerant solution is mixed in a 30-in. diameter by 30-in. high steel tank and the sodium chromate is collected in a 4-ft. × 4-ft. × 3½-ft. steel tank. The cation exchanger for removal of the sodium ions is 36-in. in diameter and 84-in. high and neoprene lined. It is filled with 20 cu. ft. of a carboxylic type resin, and provided with sampling cocks, flow meters, and a conductivity meter to measure the purity of the water, which will be equivalent to distilled water.

Summary

In comparing the two methods of handling the chromic acid anodizing solution, the ion exchange method is the more feasible. The value of chromic acid and rinse water saved is $18,762 per year (Table I) and the operating costs total as follows:

TABLE I.—Savings With the Ion Exchange Method

Item	Savings ($)
Chromic acid bath[1]	$15,732
Rinse water chromic acid	2,250
Rinse water	780
Total	$18,762

[1] Based on estimate that bath will become contaminated in one month.

Operating costs total as follows:

Regeneration chemicals	$3,005
Replacement of exchange resins	2,333
Total costs	5,550

All the chromic acid in the bath and rinse water is lost in the reduction and neutralization method. In addition, approximately $9,100 is spent for chemicals used in the treatment, making a total annual cost of $27,862.

Estimated installation costs for the ion exchanger were $35,000 and for the reduction neutralization process costs ranged from $25,000 to $35,000. Labor costs were omitted because the same amount of labor would be required for each system.

By the use of the ion exchange method the waste problem from the chromic acid anodizing bath was eliminated with a saving of approximately $22,000.

Stream Pollution

EFFECT OF SEWAGE-BORNE PHOSPHORUS ON ALGAE*

By John J. Curry and S. L. Wilson

Respectively, Chief Engineer, Connecticut State Water Commission, Hartford, Conn., and Biologist, Connecticut Agricultural Experiment Station, New Haven, Conn.

When Lake Zoar was first formed by Stevenson Dam on the Housatonic River in Connecticut there was no algae problem. However, as the years passed algae in the lake increased and by 1947 they were so plentiful that a serious nuisance was created for the property owners around the lake. At the time it was thought the most offensive part of the problem was the aquatic weeds and some efforts were made to determine the most suitable method of control. In 1949, the Housatonic Valley Clean Waters League, Inc., requested the Connecticut state government to make some studies of the problem. The problem was referred to an *ad hoc* committee comprising the agricultural experiment station, the water commission, the fish and game department and the health department, all of the state of Connecticut.

Although no funds were specifically available, the Connecticut Agricultural Experiment Station, being a research agency, conducted a general study of the problem with the support of the other interested agencies. Preliminary conclusions and recommendations for continued study were presented in a report by Mr. R. L. Holmes. Work was continued by Dr. Richard Benoit and in July, 1954, a research study was authorized.

Physical Aspects

Lake Zoar was formed behind a run-of-the-river hydroelectric plant built by the Connecticut Light and Power Company in 1919 and the drainage area of the Housatonic River at this point is about 1545 sq. mi. This company also operates a pump storage facility at Lake Candlewood. Although Lake Candlewood is not on the Housatonic River proper, the water in the lake is Housatonic River water pumped to it for storage (Figure 1).

Before entering Connecticut the Housatonic River flows through the western end of Massachusetts where its waters are used extensively by industries in the Pittsfield region. The processes of nature almost completely eliminate any pollution in the water by the time it reaches the Connecticut state line. As it flows through Connecticut the river receives little pollution before it reaches New Milford. It is an esthetically satisfactory stream and extensively used for fishing and recreation. However, there are two aspects of the water quality which are unusual for streams in this area. Flowing through limestone formations, the water is hard (75 p.p.m.) compared with other streams in the state. Total solids average about 170 p.p.m., whereas other esthetically acceptable waters in the state average 65 p.p.m.

* Presented at the 1954 Annual Meeting, New England Sewage and Industrial Wastes Assn.; Fairfield, Conn.; Oct. 28, 1954.

Limnology

Below Boardman Bridge the quality of the Housatonic River is changed materially. The major factors causing this change are: (a) raw sewage pollution from the Town of New Milford, of approximately 200,000 gal. per day; (b) the tributary Still River, which carries a large quantity of organic industrial wastes from the hat manufacturing industries in Danbury and Bethel, in addition to the trickling filter effluent of the Danbury sewage treatment plant; and (c) the Shepaug, the Pomperaug and Pootatuck Rivers which are relatively large-sized tributaries carrying no serious pollution. The Connecticut State Health Department has eliminated practically all the pollution from development adjacent to Lake Zoar. The lake volume is estimated to be 42,800 acre-ft. and the average flow of the Housatonic River at Lake Zoar is approximately 2,500 c.f.s., indicating that the lake waters are completely replaced every 9 days. During the algae season the low monthly flow (550 c.f.s.) of August and September results in a replacement of the lake volume every 40 days. However, power production causes great fluctuations in the rate of flow.

The preliminary investigation showed that during late July, as the seasonal reduction of river flow becomes apparent, it might be possible for stratification to develop. However, more detailed studies revealed that the stratification was transient and did not exist to an extent that would affect the problem.

Types of Algal Bloom

Although original complaints had specified rooted aquatic weeds as the cause of the disagreeable condition, algal blooms were an equally objectionable part of the problem. Small mats of floating algae were first observed the last of May or early June. These mats were sparsely distributed and composed of *Oscillatoria* and *Spirogyra* encrusted with diatoms. This small population was gradually replaced by *Hydrodictyon* until by the middle of July this form was exceedingly prodigious, almost completely re-

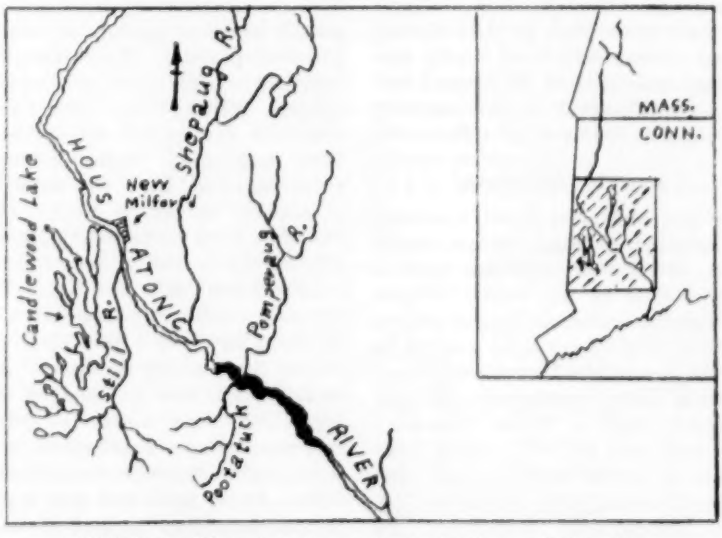

FIGURE 1.—Watershed of the Housatonic River and Lake Zoar.

FIGURE 2.—Predominating species present in an algal bloom on Lake Zoar; (A) *Microcystis aeruginosa*, (B) *Anabaena, Sp.*, (C) *Anabaena circinalis.*

placing the small former population. By mid-July *Hydrodictyon* was found in great quantities in all the shallow, quiet waters where it was intimately associated with aquatic weeds such as *Elodea*, and *Ceratophyllum*. Large mats move from place to place during changes of flow and early in the season large quantities of *Microcystis* and *Anabaena* (Figure 2) appear, usually in the upper four feet of lake water.

Corrective Measures

The first and most direct approach to alleviation of algal blooms would be the addition of algicides such as copper sulfate to the water. Under the foregoing conditions copper sulfate treatment would have to be applied to the total flow of the Housatonic River, and toxic doses maintained. It was calculated that a single treatment would cost over $15,000. Since Lake Zoar is a fairly good fishing site, continuous dosing with copper sulfate would undoubtedly depreciate its value. During long periods of time, copper tends to accumulate in the mud and has toxic effects on the organisms on which fish feed.

Laboratory experiments conducted on the effect of fungicides, organic dyes and quarternary ammonium compounds on algal growth, showed some practical promise. Even though they were more expensive per unit than copper sulfate, the concentrations necessary for control were of a much lower magnitude. Some of these organic algicides have the same order of toxicity to fish as copper sulfate, but they tend to be metabolized rapidly in the bottom muds rather than accumulating. It is possible that these algicides could be effectively applied by spraying from a boat, thereby reducing the quantity necessary. A cove in Lake Zoar was enclosed by stringing a log boom across its entrance, and experimental applications of algicides were made with inconclusive results. If an algae kill was made by the concentration applied it was obscured by the invasion of new algae.

TABLE I.—Concentrations of Phosphorus in Lake Zoar

Sample Date 1950–51	Flow (c.f.s.)	PO_4 (p.p.b.)	PO_4 (lb./day)
Sept. 2	372	33	66
Oct. 2	462	15	25
April 27	3,670	30	594
May 7	1,880	41	416
June 4	1,270	26	178
June 26	1,100	22	131
July 3	1,060	25	143
July 13	1,250	31	209
July 18	809	12	52
July 23	2,300	18	224
July 30	987	22	117
Aug. 10	312	28	47
Average	1,289	25	183

Experiments are being continued in the field on the application of various chemicals to both aquatic plants and algae.

Phosphate Aspects

Repeated tests indicated the level of nitrates and phosphates in Lake Zoar was similar to that noted in other lakes at the time of blooming. Numerous samples of the lake water have indicated that, with the materials present in the water in the dissolved state and with the fertilizing effect of the high concentrations of phosphorus and nitrogen, a bloom could be expected to be persistent. Phosphorus concentrations varied from 12 to 41 p.p.b. and averaged about 25 p.p.b. (Table I). Some members of the *Cyanophyta* (the blue-green algae) group can supply their own nitrate by fixation of atmospheric nitrogen. Phosphates, however, must be supplied from the water environment.

The volume of phosphates entering the streams in Connecticut from sewage outlets has reflected the increase noted with the increased use of synthetic detergents. Most of the sewage that is discharged to the Housatonic River in this region is given a high degree of treatment. This treatment oxidizes the organic phosphates to the mineral form available as fertilizer for the aquatic plants.

In the preliminary investigation, the bulk of the phosphates being fed into Lake Zoar came from the Still River and the Pootatuck River and the report recommended further investigation. The results of soluble phosphate determinations are shown in Table II. The values from the analysis and computation check well in view of the fact that many more individual samples were averaged for main stream values than for tributary values. Contributions of the Still River to the total phosphates available in Lake Zoar account for about 40 per cent of the total phosphates. If this source were eliminated the phosphate concentration in Lake Zoar would be reduced to 15 p.p.b. The critical concentration of phosphates for blue-green algae growth is believed to be 10 p.p.b. A reduction in phosphate should have an important effect on the algae bloom.

Lake Candlewood is filled with water taken from the Housatonic River immediately below Boardman Bridge. The soluble phosphorus content of the Housatonic River at this point during periods of high water is probably less than 33 p.p.b. However, Lake Candlewood produces no algal bloom of any consequence. Determinations made during the algae season indicate phosphorus concentrations of only 8 p.p.b. It has been suggested that the phos-

TABLE II.—Sources of Phosphorus Contributary to Lake Zoar

Location	Analyses (lb./day)	Cumulative[1] (lb./day)
Housatonic River at Boardman Bridge	98	
Sewage from New Milford	9	
New Milford industrial waste	—	
Still River	77	
Housatonic River at Rt. 133	232	184 +
Shepaug River	2	
Housatonic River above Pootatuck	167	186 +
Pootatuck River	10	
Pomperaug River	5	
Housatonic River at Lake Zoar	183	201 +

[1] Calculated.

phate is utilized early in the season by diatoms which die and sink to the bottom, removing phosphate from surface waters before the algae season. Since little additional water is pumped into Lake Candlewood during the summer the supply of nutrient is not renewed as it is in Lake Zoar.

Changing Conditions

During the past three years pollution control agencies have taken action which may affect the problem in the future. The city of Danbury is enlarging and renovating its secondary sewage treatment plant so the quantities of sewage by-passed to the Still River will be materially reduced. Six industries which discharge organic wastes from hat manufacturing directly to the Still River are constructing screens and settling basins which will keep much organic matter from entering the stream. The town of New Milford and two state institutions in the watershed are planning or constructing sewage treatment facilities.

Construction of the Shepaug Dam by the Connecticut Light and Power Company will require a reassessment of the industrial discharge at New Milford. As previously noted, minor sewage discharges around Lake Zoar have been eliminated. All these actions will reduce the levels of organic substances now discharged into the tributaries feeding Lake Zoar. As a result of efficient sewage treatment, organic phosphorus is oxidized and discharged into the streams in its inorganic form.

Phosphorus Removal

The removal of soluble phosphates from sewage treatment plant effluents through the use of alum and lime is to be studied in a pilot plant constructed with a special legislative appropriation. It is hoped that algae can be controlled by reducing the phosphorus in the water.

Preliminary studies have been made on the removal of soluble phosphate from sewage treatment plant effluents having a phosphorus content of 3.61 to 4.62 p.p.m. The treatment was in batches of 4,000 gal. with a mixing time of 10 min. and a settling time of two hours. No attempt was made to control the pH in these preliminary runs. Using 200 p.p.m. alum, 96.7 per cent of the phosphates have been removed.

Detailed experiments are planned in 1955 to show the effects of removing phosphates from a well-treated trickling filter effluent. The experimental lagoons have been filled and an attempt has been made to produce an algae bloom by inoculating one lagoon with *Microcystis* and *Hydrodictyon*. The lagoons will be operated so that seasonal algae growths will parallel those occurring in Lake Zoar.

Each lagoon is 400-ft. long by 100-ft. wide (approximately one acre) with a depth of 4.5 ft. to 8.0 ft. The measured flow of effluent from a secondary sewage treatment plant can be directed into either or both lagoons; or first treated to remove phosphate and then directed into either lagoon. The effluent is treated for phosphate removal in duplicate fill-and-draw tanks of 5,000-gal. capacity each. On a 24-hr. schedule 50,000 g.p.d. can be treated, assuming that concentrations will not be reduced below 8 p.p.b. in the lagoon. For the critical values the effluent would be diluted with the required quantity of surface water. Considering required dilution and 50,000 g.p.d. of treated effluent, flow time through the lagoons would be almost 10 days. Therefore, the range of conditions existing in Lake Zoar should be fairly well simulated.

Success in preventing algal bloom formation by stripping the soluble phosphorus from the water before it enters the lagoons remains to be fully evaluated.

RELATION OF PHOSPHORUS CONTENT TO ALGAE BLOOMS *

By Richard J. Benoit

Assistant Curator of Limnology, The Academy of Natural Sciences of Philadelphia, Philadelphia, Pa.

The problem of algae blooms is considered by the biologist in terms of the productivity of the water basin in question. Productivity merely means the number of algal cells produced in a unit time in each unit of volume or surface area of water. There can be little hope of controlling the physical factors which influence productivity, such as temperature and light intensity. The other factors relating to productivity are the dissolved chemicals essential to algal growth; such as carbon dioxide, nitrates and phosphates.

A comparison of lakes that bloom with those which do not reveals the fact that blooming lakes are more highly mineralized. What is it, from the standpoint of productivity that limits algal growth in some rather highly mineralized lakes? A simple clue comes from agriculture; phosphorus is the most common chemical limiting factor in soils. If a jar of clear lake water is brought into the laboratory and kept in the light, little algal growth will occur unless a small quantity of a phosphate is added. If nitrate is added also the growth is usually more prolific.

Lakes and rivers that are not simply sewers contain from 0 to 100 p.p.b. of phosphorus. Juday and Birge (1) give a mean of 23 p.p.b. total phosphorus for 479 lakes of northeastern Wisconsin. Hutchinson (2) found a mean of 21 p.p.b. for 23 analyses of the surface water of Linsley Pond, North Branford, Conn. Of course no phosphorus merely means an amount below the limit of sensitivity of the analytical method employed. Fortunately, a sensitive colorimetric method for phosphate is in common use and in many laboratories a zero phosphorus content means less than 1 or 2 p.p.b.

From the waste disposal standpoint the question immediately arises: how much phosphate can be added to a given lake or stream before algae blooms become imminent? Unfortunately there is no simple answer to this question even in specific cases. Another way of putting the question is: what is the critical concentration of phosphate below which algae blooms are not observed? There is no simple answer to this question.

The alert waste disposal specialist might well ask why there are no simple answers to these apparently simply questions. One reason is that the rather voluminous data relevant to the problem are inadequate because only the concentration of orthophosphate in solution is given. In most cases a filtered water sample is run through the analytical procedure and its color compared with prepared phosphate standards. The molybdate reagent for determining phosphate reacts only with the orthophosphate in solution.

This simple analytical procedure would be completely satisfactory if only orthophosphate in solution were suitable for supporting algal growth. Field observations have re-

* Presented at 1954 Annual Meeting, New England Sewage and Industrial Wastes Assn.; Fairfield, Conn.; Oct. 28, 1954.

peatedly shown good algal growth where the orthophosphate concentration of the water was either zero or very low. It is true that there is usually a high phosphate content in waters showing intense blooms. In laboratory studies using synthetic waters or culture media, it is necessary to add considerable phosphate to achieve good growth. It was reasoned that perhaps in nature algae can utilize forms of phosphorus which do not give a reaction in the routine analysis for phosphate.

Chu (3) demonstrated that pyrophosphate was not as good a source of phosphorus as orthophosphate for the marine chrysophycean alga, *Phaeocystis pouchetti*, or for the marine diatoms *Skeletonema costatum* and *Nitzschia closterium*. Pyrophosphate, however, is converted fairly rapidly to orthophosphate in dilute solution. Most green plants store a reserve of phosphate as inositol hexaphosphate. Chu showed that the calcium-magnesium salt (phytin) of this organic phosphate supported the growth of these three algae as well as, or a little better than, orthophosphate. Chu also found that organic phosphate extracted with hot water from the blades of the marine brown alga, *Laminaria* (kelp), can be utilized by *Nitzschia closterium* as effectively as orthophosphate. Harvey (4) later showed that *Nitzschia closterium* can absorb inosotol hexaphosphate and glycerophosphate equally as rapidly.

Dr. Luigi Provasoli of the Haskins Laboratory in New York in a personal communication indicated that for certain fresh water diatoms in culture free of bacteria a maintenance medium containing phosphorus only as glycerophosphate is used routinely in his laboratory. Dr. Provasoli has limited data indicating that in certain cases glycerophosphate is about ten times more effective in supporting diatom growth than orthophosphate. Most nuisance blooms are blue-green algae for which no such data are apparently available, but the fact that blue-green algae in nature grow best in waters with high organic matter suggests that these forms do grow by utilizing organic phosphate.

There is no shortage of organic phosphate in natural waters, in fact, Hutchinson (2) calculated from the data of Juday and Birge (1) that as high as 60 per cent of the total phosphate in some lakes may be in the organic form. It becomes increasingly clear, in discussing the phosphate content of natural waters, that consideration must be given to the different forms in which phosphorus exists in natural waters. These different forms are perhaps best described in terms of how they are determined analytically. Total phosphorus is determined by digesting or wet-ashing an unfiltered sample with perchloric or nitric acid and then determining the phosphate content by the usual molybdate method. If a filtered sample is digested and the sample analyzed for phosphate, the total soluble phosphate is determined. Soluble orthophosphate is determined on a filtered undigested sample. The difference between total soluble phosphate and the soluble orthophosphate is the soluble organic fraction although this term is in the strict sense a misnomer since the fraction also includes metaphosphates, pyrophosphates and other complex inorganic phosphates. These complex phosphates, which are used in modern detergent products, may also be suitable in unmodified form for supporting algal growth.

The difference between total phosphorus and total soluble phosphate is termed particulate phosphate. This form of phosphate is not immediately suitable for algal metabolism but all phosphate is potentially available when released into solution by the activity of bacteria. This is precisely what happens to the phosphate in

unsettled solids from secondary sewage treatment processes.

Another distinction that must be made before answers to the previous questions can be reasonably given is the distinction between the momentary concentration and the rate of turnover of soluble phosphate. During an algae bloom the momentary concentration of soluble phosphate may be low, but the total phosphorus will be high of necessity because of the high concentration of algal cells in the water. If phosphate is utilized by the growing and dividing cells as fast as it is introduced into the lake basin from tributaries and as fast as it is released from dead cells, the momentary concentration of soluble phosphate will remain low but the rate of turnover between bound and soluble phosphate and back to particulate phosphate can be high. Low momentary concentration and high turnover of soluble phosphate constitute the essence of what occurs during an algae bloom.

The algal utilization of organic phosphate and the possibility of a high rate of turnover during a time of low momentary concentration of soluble phosphate point out the necessity for determining all forms of phosphorus in routine analysis.

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DUES PLEASE!

Early remittance of 1956 membership dues to the Member Association Secretary will assure continued receipt of the Journal. In 1954 many members were unable to obtain a full set of Journals because of failure to renew their membership promptly. Delinquent members' names are removed from the mailing list prior to the mailing of the February issue of SEWAGE AND INDUSTRIAL WASTES. Reinstatement is time consuming and costly to the Federation and its members. In addition, each extra handling of a member's record multiplies the chance for error.

To insure efficient service—pay dues promptly! Many thanks!

STREAM BIOTA AND STREAM POLLUTION

By Charles B. Wurtz

Consulting Biologist, Philadelphia, Pa.

The interpretation of stream conditions based on the population of plants and animals in the stream has been used for more than 50 years. This interpretation is based on an analysis of the organisms present. Some species are known to be tolerant to varying degrees of pollution, while others are recognized as non-tolerant. Earlier workers recognized that when polluted conditions prevailed in a stream, and the non-tolerant species were killed or driven out, the tolerant species increased their populations enormously. These tolerant species were then considered indicators of pollution. However, the presence of tolerant species in moderate or few numbers cannot be considered as an indication of pollution. These species are a component part of every diverse stream population. Populations, not species, reflect stream conditions. The absence of non-tolerant species is of far greater significance than the presence of tolerant species.

Stream Classification

Kolkwitz and Marsson (1) proposed a classification of streams based on the organisms found to be present. This concept of stream classification has been developed further by subsequent workers. The most satisfactory method was presented diagrammatically by Bartsch and Churchill (2). A modified diagram of their method is presented in Figure 1. The most important feature of this method of stream classification is the recognition of two intermediate conditions between clean and polluted waters. These are the zone of degradation and the zone of recovery. This classificatory arrangement is now commonly used (3)(4)(5). Other classificatory schemes have not taken into consideration the two kinds of intermediate zones between the clean and polluted zones. In Figure 1 a functional classification of pollutants is presented, and the pollution zone would be classified according to these three basic types of pollutants.

As every stream is individualistic the extent of each zone varies from stream to stream. Seasonal changes, differences in volume, type or intensity of the pollutant, differences in the size of the stream, etc., all influence the extent, presence or absence of the various zones. In lakes that receive a pollutant the zones may occur as concentric rings emanating from the point of entry of the pollutant. In large streams (or with small volume effluents) the zones may

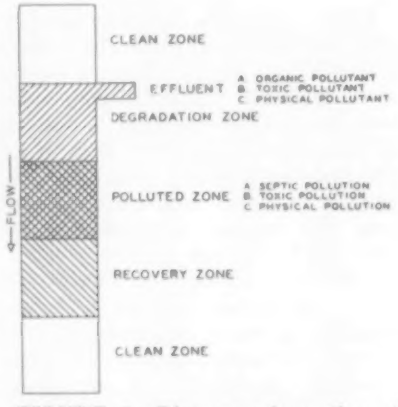

FIGURE 1.—Diagrammatic section of a stream affected by the entry of a pollutant.

occur on only one side of the stream. In many cases a pollutant does not cause a pollution zone; *i.e.*, a mild pollutant may depress the stream biota to cause a degradation zone, but this may pass directly into a recovery zone.

Methods

In a comprehensive study of the Conestoga Creek, Lancaster County, Pennsylvania, by the Academy of Natural Sciences of Philadelphia * in 1948, a great deal of data on the biota were accumulated from the various streams of that drainage basin. In the report (6) submitted on that survey the data were presented graphically as a series of histograms. The method used for the construction of these histograms was subsequently published (7)(8)(9). The method was also applied to the Red Clay Creek by Kaplovsky and Harmic (10).

In this method each histogram included seven columns on the basis that the members of certain groups responded similarly to a given environment. Accordingly, the first two columns included pollution tolerant organisms while the remaining five columns were simply arranged in a phylogenetic sequence. No recognition was given to pollution tolerant species that belonged to the insects, fish, or other phylogenetic groups not recognized in the first two columns. Furthermore, no recognition was made of the diversity of life-forms found within any one group (adult beetles *vs.* larval beetles). In interpreting the histograms reference was made to the relative height of each column in relation to a 100 per cent line. This line represented the average population level of 9 healthy stations in the Conestoga Basin. Although this was a useful device, and justified within the limits of the study, the same "average" obviously cannot be used in studying other streams.

If any column of a histogram is intended to reflect a degree of tolerance, then all the columns should do so. Moreover, to present all the columns as simply phylogenetic groups does not permit a proper analysis of the population present in relation to stream conditions. Both these objections are overcome by the following method.

Basic Life-Forms

The method presented here for streams and rivers includes four columns in each histogram. Each column includes all the organisms which share a basic life-form. Biologically all free-living organisms can be reduced to five basic life-forms. These are:

1. The burrowing organisms, which include all organisms that bury themselves in any substrate.
2. The sessile organisms, which include all organisms that are fixed to the surface of a substrate, as well as small animals of very limited motility, such as the hydras.
3. The foraging organisms, which include all organisms that actively browse on the bottom organisms, but which are rarely free swimming.
4. The pelagic organisms, which include active, free-swimming organisms.
5. The plankton organisms, which include all organisms of limited motility which depend upon water currents for transportation. Usually minute, true plankters are not found in running waters, and are not discussed further. In a lake or estuarine habitat they would be included as a fifth column in the histograms of the method presented here.

In the studies leading to this work it was found that certain groups of organisms did not yield information of enough value to justify their inclusion in a biological stream survey.

* The author was a staff member of the Academy and participated in the survey and the preparation of the report.

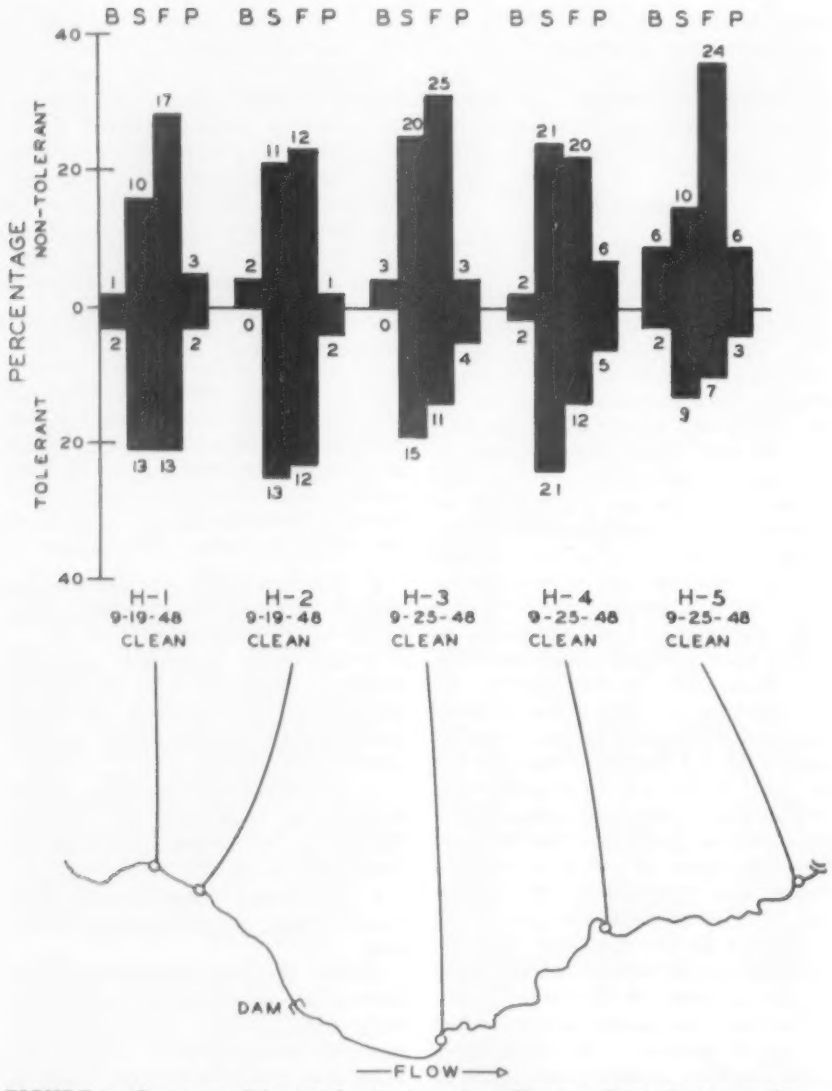

FIGURE 2.—Stream conditions at five stations along Hammer Creek, Lancaster County, Pa., Sept., 1948.

These were the microscopic animals which are often represented by a great many species in a particular habitat. All of these animals have that peculiar faculty, common to microscopic species, which enables them to form tremendous populations in short periods of time, measurable in hours. Further, they can disappear from the faunal complex just as quickly. Because of this abbreviated life history they are not, at the species level, a stable element in the faunal composition.

Repetitive surveys show a wide fluctuation in the species diversity of such organisms as a result of this instability. Therefore these animals, which include the protozoa, non-tri-

clad turbellaria, aschelminthes, and entomostracan crustacea, have been deleted from the data forming the basis of the examples that follow. Other organisms (sponges, bryozoans, hydroids, naid worms, and arachnoids) not incorporated in the histograms of the Conestoga Basin report are included here. Though not included originally, because of their sporadic occurrence, they do contribute to the faunal picture.

Figures 2 through 4 illustrate the proposed method of presenting data on the biota of a stream. In these figures each station is represented by a histogram of four columns projected above and below a baseline. That portion of the column above the baseline represents the non-tolerant organisms, while the part below the baseline represents the tolerant organisms. The columns are plotted as a frequency index in which the total number of species found at any station represents a frequency of 100 per cent for that station. Thus the contained area in all the columns of any one histogram equals 100 per cent. The absolute number of species found that contribute to the non-tolerant and tolerant groups is presented above and below each column, respectively. The four columns of each histogram are indicated (from left to right) as B (burrowing organisms), S (sessile organisms), F (foraging organisms), and P (pelagic organisms). In addition to the histograms, each figure includes a map showing the location of each station and the date it was surveyed.

Interpretation

Since the data are expressed as a frequency index it is apparent that they represent a continuous variate. Therefore the interpretation of the

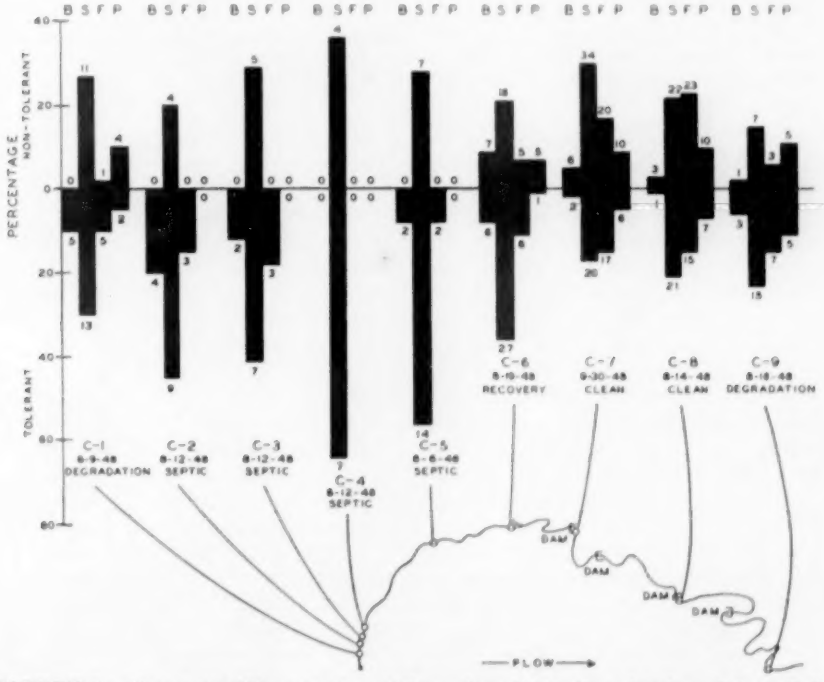

FIGURE 3.—Stream conditions at nine stations along Lititz Run (stations C-1 to C-6) and Conestoga Creek (stations C-7 to C-9), Lancaster County, Pa., Aug.–Sept., 1948.

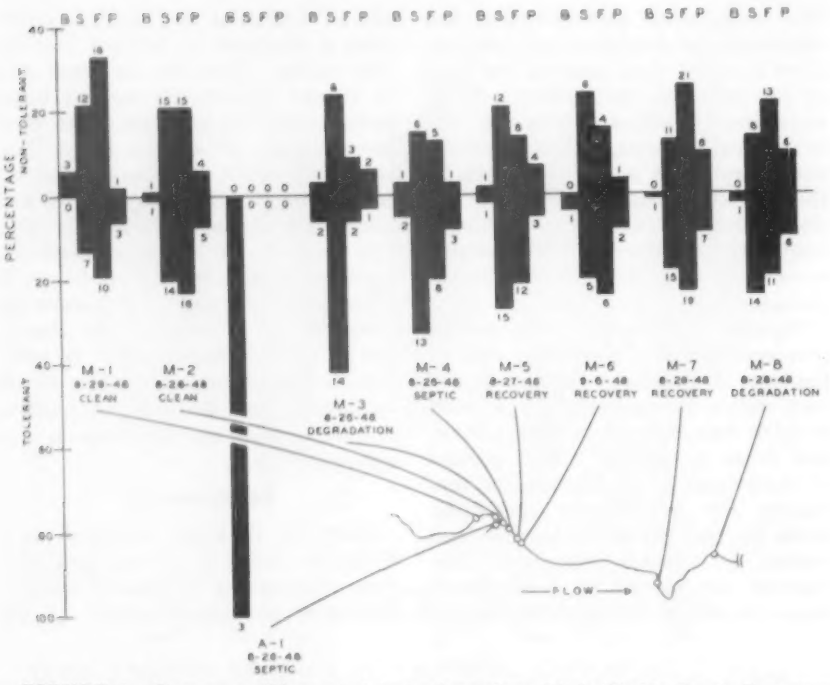

FIGURE 4.—Stream conditions at eight stations along Little Muddy Creek, Lancaster County, Pa., Aug.–Sept., 1948. Station A-1 reflects conditions in an industrial effluent entering the stream between stations M-2 and M-3.

stream condition is derived from the relationship between two variables. These are the relative proportion of non-tolerant to tolerant species, and the structure of the population as presented by the four columns of each histogram. As with all continuous variates, borderline cases exist. In the interpretation of stream conditions it is of the utmost importance to recognize environmental influences that function independently of possible pollution. The interpreter should have first hand knowledge of the stream; and must have comprehensive descriptive notes of the station concerned.

In general, the stream may be considered as a clean-water stream when the non-tolerant species represent more than 50 per cent of the population. If the non-tolerant species drop much below this level there is cause for concern as regards stream conditions. The factor that causes this depression can be interpreted as pollution.

The relative proportion of the population composing each column of life-forms would be, for the normal, clean-water station, approximately 5 per cent, 40 per cent, 45 per cent and 10 per cent, respectively. However, this is considerably modified by environmental conditions as well as by the effects of pollution. Nevertheless, the proper interpretation of the relative species diversity in each column is the most sensitive method of evaluating stream conditions.

Unless gross differences exist between the total number of species of any one station compared to any other station of a stream, this feature cannot be used for the interpretation of pollution. It may, however, be con-

sidered as supporting evidence for the conclusions drawn as regards any particular station. The clean stations studied ranged from 53 species to 115 species in total species diversity. This difference was chiefly a function of stream size. The polluted stations studied ranged from 3 to 46 species. This difference was a function of both stream size and severity of pollution. The range of species diversity of both clean and polluted stations was widely overlapped by the range of species diversity of the recovery and degradation zones. These latter, intermediate conditions ranged in total species diversity from 25 to 75 species.

Material Studied

In the following examples from the Conestoga Basin three series of stations are shown. In each case the station nearest the headwaters is shown to the left in the figure.

The first series of stations is presented in Figure 2. The five stations (H–1 through H–5) were distributed along Hammer Creek, which was a clean stream with some organic enrichment.

Stations H–1 and H–2 have 50 per cent and 49 per cent non-tolerant species, respectively. This is the minimum for clean-water stations. This condition is attributed to some local environmental influence that was not recognized at the time the survey was made.

Station H–2 was not adequately seined at the time of the survey because of obstructions in the stream bed. Had adequate seine sampling been done the number of pelagic organisms undoubtedly would have approached the numbers found at stations H–1 and H–3.

The data used in the construction of the histograms are presented in Table I.

The second series of stations is presented in Figure 3. Station C–1 was near the source of Lititz Run and here

TABLE I.—Distribution of Tolerant and Non-Tolerant Species in Hammer Creek, Lancaster County, Pa., Aug.–Sept., 1948

Species	Stations				
	H-1	H-2	H-3	H-4	H-5
Non-tol. burrowing					
No.	1	2	3	2	6
Per cent	2	4	4	2	9
Tol. burrowing					
No.	2	0	0	2	2
Per cent	3	0	0	2	3
Non-tol. sessile					
No.	10	11	20	21	10
Per cent	16	21	25	24	15
Tol. sessile					
No.	13	13	15	21	9
Per cent	21	25	19	24	13
Non-tol. foraging					
No.	17	12	25	20	24
Per cent	28	23	31	22	36
Tol. foraging					
No.	13	12	11	12	7
Per cent	21	23	14	14	10
Non-tol. pelagic					
No.	3	1	3	6	6
Per cent	5	2	4	7	9
Tol. pelagic					
No.	2	2	4	5	3
Per cent	3	4	5	6	4
Total non-tol.					
No.	31	26	51	49	46
Per cent	50	49	63	55	69
Total tol.					
No.	30	27	30	40	21
Per cent	50	51	37	45	31
Total number	61	53	81	89	67

the stream was in a degradation zone due to organic pollution. This condition gave way to septic pollution at station C–2 which prevailed through station C–5. Station C–4, with a population composed of only 11 sessile organisms (algae), was the most severely injured stretch of the stream. The absence of any pelagic forms from stations C–2 through C–5 is indicative of severe pollution. At station C–6 the stream entered a recovery zone and immediately below this station the stream entered the Conestoga Creek. Stations C–7 and C–8 were large (159 and 136 ft., wide respectively) and had a large species diversity (115 and 102 species, respectively). The stream

at these stations was clean. At station C–9 the overflow from the Lancaster sewage system reduced the stream to a degradation zone.

The data used in the construction of the histograms are presented in Table II.

The third series of stations is presented in Figure 4. Station M–1 was above Adamstown on Little Muddy Creek and station M–8 was near the mouth of Little Muddy Creek. In this series of stations along Little Muddy Creek one station (A–1) was not part of the stream proper. This station, which was grossly polluted, was the effluent canal carrying wastes from a hat factory. It entered the stream immediately above station M–3. Dilution was effected here and station M–3 was in the zone of degradation. This gave way to a pollution zone at station M–4. The stream was in a recovery zone from station M–5 through station M–6. At station M–7 the stream was within the limits of a clean stream (49 per cent non-tolerant species), but the relative proportions of the various columns were somewhat skewed. Burrowing organisms were still few in number (1 per cent). The high percentage (19 per cent) of pelagic forms (including 10 species of fish) indicated a higher than average food supply resulting from the organic enrichment farther upstream. This station is considered

TABLE II.—Distribution of Tolerant and Non-Tolerant Species in Lititz Run and Conestoga Creek, Lancaster County, Pa., Aug.–Sept., 1948

Species	Stations								
	C-1	C-2	C-3	C-4	C-5	C-6	C-7	C-8	C-9
Non-tol. burrowing									
No.	0	0	0	0	0	7	6	3	1
Per cent	0	0	0	0	0	9	5	3	2
Tol. burrowing									
No.	5	4	2	0	2	6	2	1	3
Per cent	12	20	12	0	8	8	2	1	6
Non-tol. sessile									
No.	11	4	5	4	7	16	34	22	7
Per cent	27	20	29	36	28	21	30	22	15
Tol. sessile									
No.	13	9	7	7	14	27	20	21	15
Per cent	30	45	41	64	56	36	17	21	33
Non-tol. foraging									
No.	1	0	0	0	0	5	20	23	3
Per cent	2	0	0	0	0	7	17	23	6
Tol. foraging									
No.	5	3	3	0	2	8	17	15	7
Per cent	12	15	18	0	8	11	15	15	15
Non-tol. pelagic									
No.	4	0	0	0	0	5	10	10	5
Per cent	10	0	0	0	0	7	9	10	11
Tol. pelagic									
No.	2	0	0	0	0	1	6	7	5
Per cent	5	0	0	0	0	1	5	7	11
Total non-tol.									
No.	16	4	5	4	7	33	70	58	16
Per cent	39	20	29	36	28	44	61	57	35
Total tol.									
No.	25	16	12	7	18	42	45	44	30
Per cent	61	80	71	64	72	56	39	43	65
Total number	41	20	17	11	25	75	115	102	46

TABLE III.—Distribution of Tolerant and Non-Tolerant Species in Little Muddy Creek, Lancaster County, Pa., Aug.–Sept., 1948

Species	Stations								
	M-1	M-2	A-1	M-3	M-4	M-5	M-6	M-7	M-8
Non-tol. burrowing									
No.	3	1	0	1	1	1	0	0	0
Per cent	6	1	0	3	3	2	0	0	0
Tol. burrowing									
No.	0	1	3	2	2	1	1	1	1
Per cent	0	1	100	6	5	2	4	1	2
Non-tol. sessile									
No.	12	15	0	8	6	12	6	11	8
Per cent	22	21	0	24	15	21	24	13	14
Tol. sessile									
No.	7	14	0	14	13	15	5	15	14
Per cent	13	20	0	42	33	27	20	18	24
Non-tol. foraging									
No.	18	15	0	3	5	8	4	21	13
Per cent	33	21	0	9	13	14	16	26	22
Tol. foraging									
No.	10	16	0	2	8	12	6	19	11
Per cent	19	23	0	6	20	21	24	23	19
Non-tol. pelagic									
No.	1	4	0	2	1	4	1	8	6
Per cent	2	6	0	6	3	7	4	10	10
Tol. pelagic									
No.	3	5	0	1	3	3	2	7	6
Per cent	6	7	0	3	8	5	8	9	10
Total non-tol.									
No.	34	35	0	14	13	25	11	40	27
Per cent	63	49	0	42	33	45	44	49	46
Total tol.									
No.	20	36	3	19	26	31	14	42	32
Per cent	37	51	100	58	67	55	56	51	54
Total number	54	71	3	33	39	56	25	82	59

to be transitional between the recovery zone and a clean-water station.

At station M–8 Little Muddy Creek entered a mild zone of degradation. This condition was brought about by siltation which, ultimately, would lead to physical pollution.

The data used in the construction of the histograms are presented in Table III.

Acknowledgment

The author appreciates the courtesy of the Sanitary Water Board of Pennsylvania for permission to use data from the survey of Conestoga Creek, Pennsylvania.

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WONDER OF AMERICAN ENGINEERING

Tribute to the unsung sanitary engineer in the sewage and industrial wastes field is the recent inclusion of one of his projects among the Seven Wonders of American Engineering. The American Society of Civil Engineers recently announced the selection of the Chicago sewage disposal system as one of these wonders ranking with the Colorado River Aqueduct, Empire State Building, Grand Coulee Dam, and Columbia Basin Project, Hoover Dam, Panama Canal, and the San Francisco-Oakland Bay Bridge. The ASCE choice was three years in the making. There are countless projects that have been hailed more or less legitimately as one of our civilization's seven wonders.

In solving the problems of towns and cities, water supply requirements are naturally paramount. The proper disposal of human wastes has often been put off until attention has been forced to this vital need. Development of the Chicago sewage disposal system, operated by the Sanitary District of Chicago, evolved from such dire necessity.

Pollution of Lake Michigan with Chicago's own wastes jeopardized the city's water supply. As a result the Chicago Drainage Canal was dug (no small task in itself) and then the sewage treatment facilities were placed in operation in 1938 to reduce pollution of the Des Plains and Illinois rivers.

Largest treatment plant in the world is the West-Southwest Plant, with a diffused air activated sludge system, capable of treating 900 million gallons per day—equivalent to a population of 2,833,000. The three Chicago plants, served by a vast combined sewer system, can handle wastes from a population of 4,320,000 plus industrial wastes with a population equivalent of 3,000,000. These plants treat 1,100 million gallons per day and their cost has been estimated at 310 million dollars.

Everyone in the waste treatment field can take pride in some reflected glory from this recognition.

ESTIMATING STREAM POLLUTION SURVEY COSTS

By Linvil G. Rich and David A. Jones

Respectively, Associate Professor of Sanitary Engineering † and Graduate Student,‡ Virginia Polytechnic Institute, Blacksburg, Va.

In the past decade, stream pollution surveys have become standard activities in the field of sanitary engineering. Both industry and municipality are making wide use of this type of survey as one of the first steps in the determination of waste treatment requirements.

Generally speaking, the economics of stream pollution surveys have been ignored. Although costs of equipping analytical laboratories of specific types and sizes can be found in the literature, little information is available to aid management either in estimating the cost of a proposed stream pollution survey or in applying cost accounting procedures after the survey has been completed.

This article describes a rational method for computing the cost of stream pollution surveys. Circumstances under which each survey is made range widely and the costs as computed by the method described may not prove accurate for any particular survey or even any particular test. It is believed that the method, if modified to meet specific field conditions and laboratory organization, will provide a reasonable estimate of cost.

Method of Analysis

The stream survey has been divided into two major divisions: (a) field operation and (b) laboratory operation. These operations, in turn, have been subdivided into unit operations. Within each unit operation the labor and equipment requirements have been computed for cost estimation. The total cost of the survey, then, will be equal to the sum of the unit operation costs plus the cost of overhead.

Field Operation

The field operation consists of two activities: (a) travel to and from the stream sampling points and (b) the sampling operation. Travel to and from the sampling points may be accomplished by using either a truck or a boat, or both, depending on the accessibility of the stations. The total cost of the field operation, therefore, may be determined from the following expression:

Total Cost of Field Operation	=	Operating Cost of Travel Activity	+	Labor Cost of Travel Activity	+	Operating Cost of Sampling Operation	+	Labor Cost of Sampling Operation

or

$$C_f = [M_t O_t + M_b O_b] + \left[W_s P \left(\frac{M_t}{V_t} + \frac{M_b}{V_b}\right)\right] + \left[D_s \frac{S_n}{S_s}\right] + \left[\frac{P T_s S_n W_s}{60}\right]$$

† Presently, with the USPHS, U.S.A. Operations Mission to La Paz, Bolivia.

‡ Presently, U. S. Air Force.

where

C_f = total cost of field operation ($).
D_s = depreciation and maintenance on sampling equipment per year ($).
M_b = total boat travel (mi.).
M_t = total truck travel (mi.).
O_b = operation, maintenance, and depreciation on boat motors, boat trailer, and other boat equipment ($ per mi.).
O_t = operation, maintenance, and depreciation on truck ($ per mi.).
P = personnel making sampling trip (no.).
S_e = estimated number of samples collected per year.
S_o = samples collected per operation (no.).
T_s = average time spent at each sampling station (min.).
V_b = average speed of boat travel (m.p.h.).
V_t = average speed of truck travel (m.p.h.).
W_s = average wage rate of sampling personnel ($ per hr.).

The values of most of the variables in the preceding equation would be established by conditions imposed by the survey. The total operations costs for the boat and truck, the depreciation and maintenance on the boat equipment, and the speed of boat and truck travel, however, should not vary significantly from survey to survey. Results of a recent study (1) indicate that under most conditions the following values are reasonable approximations for these variables:

O_b = $ 0.35 per mi.
O_t = $ 0.08 per mi.
D_s = $10.00
V_b = 5 m.p.h.
V_t = 35 m.p.h.

Normally it would be more convenient to compute the survey cost on the basis of per trip operations. In such cases, the total cost of the field operation would be the cost per field trip multiplied by the number of field trips comprising the survey. If any of the variables might be expected to change materially during the course of the survey, it may be easier to compute directly the total cost of the field operation for the complete survey.

Laboratory Operation

The total cost of the laboratory operation for any stream survey is equal to the sum of the costs involved in performing the individual analytical tests. By resolving each test into its cost requirements for chemicals, apparatus and labor, the following expression for the total cost of laboratory operations per test is developed:

$$\text{Total Cost of Laboratory Operations per Test} = \text{Cost of Chemicals} + \text{Depreciation on Apparatus} + \text{Labor Costs}$$

or

$$C_l = [C_i + N\,C_{ps}] + \left[D_a \frac{N}{N_e}\right] + \left[\frac{T_r W_e + N\,T_a W_{el} + N T_e W_{clu}}{60}\right]$$

where

C_l = total cost of laboratory operations per test ($).
C_i = fixed cost of chemicals ($).
C_{ps} = per sample cost of chemicals ($).
D_a = apparatus depreciation cost ($ per yr.).
N = samples analyzed (No.).

N_s = estimated number of samples analyzed per year.
T_a = average analysis time per sample (min.).
T_c = average cleanup time per sample (min.).
T_r = average time spent preparing reagents (min.).
W_c = wage rate of chemist ($ per hr.).
W_{ct} = wage rate of chemist or technician ($ per hr.).
W_{ctu} = wage rate of chemist, technician, or utility man ($ per hr.).

A detailed study was made (1) of 32 analytical tests which have been employed in connection with stream pollution surveys. The chemical and apparatus requirements for each test were determined, for the most part, from procedures outlined in *Standard Methods for the Examination of Water and Sewage* (2). Costs of chemicals and equipment were taken from the 1954 catalogs of two major laboratory supply houses.

The chemical costs were separated into fixed and per-sample components. It was assumed that a nominal quantity of reagent, independent of the number of samples tested, would be prepared at the beginning of each test or series of tests. The fixed cost, then, is the cost of chemicals required to prepare reagents in the quantities (usually liters) suggested in *Standard Methods for the Examination of Water and Sewage*. Some loss will occur in performing a particular test on a large number of samples when more reagent is needed than is provided for in the fixed cost of chemicals. Loss here, however, will be compensated, to a degree, by the savings realized when reagents can be stored and used again. In many instances the addition of a definite quantity of chemical or reagent will be specified for a given analysis. In cases where that particular chemical or reagent is not made up ordinarily in the laboratory, but is a commercially prepared product, the per-sample cost was computed.

Where a piece of equipment was rather expensive and could be used in performing several tests, the cost was assigned to the test or tests, in which that piece of equipment serves a prominent function. As an illustration, although the analytical balance is employed in every test where standard solutions are required, the cost of the balance was charged to the solids tests were it is used as a key piece of equipment. Relatively inexpensive items such as pipettes, burettes, flasks, cylinders, color tubes, etc. were charged off completely to each test where they are required. All laboratory equipment, other than glassware, was amortized on a 10 yr. basis. Glassware was charged off in two years.

For each test, in order to establish a depreciation charge for laboratory apparatus on a per-sample basis, an estimate must be made of the number of samples to be analyzed during that year. Such an estimate is difficult to make unless it is based on a contemplated program of analyses or upon analyses performed in prior years. In many instances, however, even a sizeable error in the estimation of this variable will not affect radically the total cost of the analysis.

Estimates of the time requirements for the preparation of reagents, analysis of samples, and clean-up operations were derived from time studies made by the authors and by Weston *et al.* (3). In general, time requirements are based on procedures outlined in *Standard Methods for the Examination of Water and Sewage.*

The fixed and per-sample costs of chemicals, the total depreciation costs of apparatus per year, and the labor requirements for the 32 tests considered are presented in Table I.

Overhead Costs

Several expenses incurred in carrying out a stream survey cannot be

TABLE I.—Analysis of Laboratory Expenses

Test	Costs					
	Labor			Chemicals		Apparatus
	T_r (min.)	T_a (min.)	T_s (min.)	C_t ($)	C_{ss} ($)	D_a ($)
Acidity	40	5	5	0.01	Negl.	11.81
Alkalinity	40	5	5	0.02	Negl.	12.44
Aluminum	35	15	9	0.43	None	20.67
Bacteriological Examination	75	15	38	1.15	None	144.47
Biochemical Oxygen Demand	140	50	20	5.12	Negl.	155.72
Chemical Oxygen Demand	55	30	9	0.43	0.50	46.64
Chlorides	80	8	10	0.47	0.01	17.76
Chlorine Demand	105	18	19	1.39	None	36.82
Chromium	55	18	18	0.25	0.07	24.44
Color	25	14	10	4.37	None	19.42
Copper	110	14	19	1.15	None	23.99
Dissolved Oxygen	100	4	5	5.05	Negl.	16.54
Grease	10	30	16	None	0.48	38.62
Hardness	10	10	4	None	0.07	8.27
Hydrogen Ion Concentration	10	4	3	None	0.07	58.75
Iron	70	25	16	0.48	None	11.46
Microscopical Examination	†	†	†	None	0.49	90.85
Nitrogen: Ammonia	100	9	14	4.29	None	22.99
Nitrate	100	18	22	2.00	Negl.	35.71
Nitrite	65	11	11	1.51	None	17.79
Total Organic	130	20	21	5.57	0.04	28.32
Odor	20	15	19	None	None	30.28
Oil	10	30	15	None	0.23	9.59
Phenols	160	25	20	0.22	Negl.	39.30
Phosphate	80	20	16	1.48	0.13	25.23
Silica	55	14	14	0.31	None	20.24
Solids: Suspended	20	30	5	None	None	55.33
Total	10	30	3	None	None	51.10
Sulfates	90	15	10	0.77	0.11	16.46
Sulfides	90	8	18	4.60	None	20.30
Toxicity to Fish	30	90	11	None	5.15	73.98
Turbidity	45	10	6	0.01	None	21.30

† Labor requirements vary considerably depending upon the type and ability of personnel performing the test.

charged directly to any of the component operations; office equipment and supplies, clerical work, supervision and administration. These and other indirect expenses must be included as another element of cost—that of overhead. Since overhead is a function of the organizational structure and its size, and of operational procedures of the laboratory, it is impossible to estimate the indirect expenses without detailed information concerning these variables. Overhead may be charged off as a percentage of the total direct costs.

Profit and Professional Fee

When the stream survey is supervised and conducted by a commercial testing laboratory, profit and professional fees would be added to the cost of the survey. As a rule business ventures offering technical services compute their profit as a percentage of total expenses (materials, labor, and overhead). The percentage will vary with the business risk, the laboratory investment, the competition and even the personal opinions of the management.

The professional fee is the charge made for professional services rendered. The type of professional services involved in the performance of stream pollution surveys commonly includes the determination of the scope of survey, the selection of tests to be performed, the establishiment of sampling stations and sampling schedules, the analysis of test data, and the preparation of a report containing conclusions and recommendations. Compensation for professional services is based on the nature, magnitude, and complexity of the survey work, as well as the character of the services rendered.

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SANITARY ENGINEERING DEGREES—1954 *

There were 164 degrees awarded to graduates from undergraduate sanitary engineering courses in 1954. All but 8 of the 40 colleges and universities reporting the availability of undergraduate sanitary engineering courses had graduates. In 1953 the number of graduates was 216. The average number of graduates per year for the last 5-year period (1949–53) was 242.

The number of sanitary engineering graduates per 1,000 engineering graduates in 1954 was 7.5. In 1953 and in the 5-year period 1949–53, there were 9.0 and 6.2 sanitary engineering graduates per 1,000 engineering graduates, respectively.

Master's degrees in sanitary engineering were granted to 120 candidates in 1954; 25 of these were nationals of foreign countries. Of the 56 institutions offering graduate work in sanitary engineering at this level, 30 (or 54 per cent) had graduates. Eight universities gave 75 of the 120 degrees, and 22 other universities awarded the remaining 45 degrees. The number of such degrees granted in 1953 was 102; 20 of these were earned by foreign nationals. The average number of master's degrees granted per year for the last 5-year period (1949–53) and the last 10-year period (1944–53) was 129 and 108, respectively.

The nine doctor's degrees awarded in 1954 by five institutions were all to nationals of the United States. Twenty-one other universities offering work at this level had no successful candidates this year. In 1953, five doctor's degrees, three of which were to foreign nationals, were granted. In the last 5-year period (1949–53), the average number of doctor's degrees conferred per year was 6; in the last 10-year period (1944–53) the average was 4.

* From—"Master's and Doctor's Degrees in Sanitary Engineering in 1954" and "Graduates from Undergraduate Sanitary Engineering Courses in 1954" by A. P. Miller, USPHS, Washington, D. C.

POTENTIAL CONFLICTS BETWEEN SUPPLEMENTAL IRRIGATION AND POLLUTION ABATEMENT PROGRAMS

By John W. Wiseman

Public Health Engineer, Robert A. Taft Sanitary Engineering Center, USPHS, Cincinnati, Ohio

There is an increasing interest in the use of supplemental irrigation to increase crop production in areas where previous practice has depended entirely on normal rainfall. References in the literature indicate a growing appreciation of the benefits from irrigation systems and an increasing number of installations. This presentation is most pertinent to those areas where presently developing and potential supplemental irrigation uses and needs are not accompanied by adequate attention to potential conflicts in water uses. Generally, these areas are located in the more humid sections of the nation as distinguished from the arid sections where irrigation is a prerequisite to diversified crop production.

It is recognized that there are no definite lines of demarcation, either as to area or rainfall, between arid and humid areas. Annual variations in climatic conditions would preclude sharp lines of demarcation even though it were otherwise possible to make them. The areas of potential conflict considered here are more likely to occur where total water supply has been relatively plentiful with respect to the total water needs, and where limited conflicts in the past between water uses have not necessitated comprehensive consideration of water use control. The subsequent discussion of irrigation water uses and needs refers specifically to such areas.

Use of water for irrigation is largely a consumptive use. The potential effect on natural stream flow, should the practice become general or even substantial in any given area, leads to concern over effects on other water uses. State, interstate and federal water pollution control agencies are particularly concerned with possible effects on water pollution control programs. In practically all instances, critical low flows in streams have direct and governing influence on polulution abatement requirements. Reduction in low flows resulting from unforseen consumptive uses, such as supplemental irrigation, could have serious effects on pollution control programs. These effects could extend beyond state boundaries and become of interstate and regional interest. It is desirable to emphasize again that these statements apply specifically to the eastern, or more humid part of the Nation.

For at least a century there has been need for appropriation of water in the western states. This need led to the development, over the years, of systems to appropriate water rights (1). The 17 western states and the State of Minnesota operate under the appropriation doctrine or a combination of this doctrine and that of riparian rights (2). Regulations for appropriation are not uniform and further changes can be expected. However, in nearly all the western states some form of application to state agencies to appropriate water is required and provisions are made for assistance in adjudication of water rights by state agencies. Also, there are numerous interstate agreements

or compacts relating to the use of water. In contrast to the western states the eastern states operate under a system of riparian rights. State agencies to administer water use control are largely non-existent, advisory only, or are limited as to scope of interests. It is a reasonable conclusion that, up to this time, conflicts in demand for the total water use have not been sufficient to bring about comprehensive development of principles for allocation of water or establishment of state agencies to administer them. There are indications that the pressures of increasing water use are bringing the attention needed for additional legislation. Recently it has been reported that about 25 of the eastern states have enacted or have under consideration some form of legislation related to control of water uses (10). In general, this legislation does not take into consideration the interstate aspects of water-use control.

Significance of Supplemental Irrigation Potential in Humid Areas

It has been reported that, as of 1945, the total irrigated area in the 17 western states was 19,500,000 acres and in the eastern states about 1,100,000 acres (3). In 1952 the statement was made that in the last 10 years the proportionate growth of irrigated lands was five times as fast in the East as in the West (4).

A bulletin of the Indiana Flood Control and Water Resources Commission(5) states: "The artificial watering or supplemental irrigation of growing crops is increasing in Indiana. . . . The use may become popular . . . to assure the correct amount of moisture during a critical portion of the growing season. . . . a rapid increase (in use of water for irrigation) is expected."

The irrigated acreage in the Ohio River basin in 1950 has been estimated at 20 times the area irrigated in 1944. About 60 per cent of the irrigation water comes from surface supplies (6).

The preceding references are a representative cross section of views on the potential increase in the use of water for irrigation in the relatively humid part of the United States. Without exception the prediction is that there will be an increased demand for irrigation water. The following quotation by the President's Water Resources Policy Commission (6) indicates substantial justification for these predictions: "Various preliminary experiments carried out in nearly every state within the (Ohio) basin indicate that supplemental irrigation will produce large, in some cases, phenomenal, increases in yields of crops and pastures."

Relation of Area Irrigated to Consumptive Water Use

Several factors affect the amount of water consumption for irrigation per unit of area. The type of crop, the climate (particularly variations in climate in areas where irrigation is regarded as a supplement to natural water supply), the type of irrigation, the topography, the type of soil and the available water supply are factors which have a bearing on the quantity used. Individual decisions on water application, influenced greatly by the availability and cost of water, can be the most important factors governing the quantity of water used. In those cases where natural stream flow rather than stored water is used, the short term peak uses during critical periods are more significant than the total seasonal use. Moreover, the per cent of total irrigated land being irrigated at any time would be quite important. For example, if one concentrated irrigation period per week is sufficient, the extent to which the demands for the total area are uniformly distributed would affect the instantaneous rate of withdrawal from natural stream flow.

Present practice in humid areas indicates preference for sprinkler type irrigation. With this system it is probable that less total water is pumped but a higher percentage of the amount pumped will be consumed. Different views have been expressed on the amount of water required. The concensus seems to be that one inch per week, usually delivered in a single application, is sufficient for average loam soils and as much as two inches per week may be required for sandy soils (7). Irrigation farmers in Ohio reportedly use from three to five inches of water per season, or an average of about 100,000 gal. per acre (8).

Effect on Stream Flows

To demonstrate possible effects of extensive development of irrigation the Maumee River basin (Ohio-Indiana-Michigan) will serve for an area to be considered. The topography is of low relief, suitable for irrigation and a large percentage of the area is devoted to agriculture (approximately 90 per cent). In addition, a recent water pollution control report has been prepared including estimates of critical low stream flows and pollution abatement requirements (9). No investigation has been made as to the existing or potential irrigation developments in the basin. However, it is an area where the development of irrigation could have a substantial effect on other water uses.

The area of the basin is about 6,586 square miles (4,215,000 acres). Information from the previously mentioned report on stream flows is as follows:

Stream	Location	Critical Flow (c.f.s.)[1]	Drainage Area (sq.mi.)	Yield Per Sq.Mi. (c.f.s.)
Auglaize	Ft. Jennings, Ohio	11	333	0.033
Blanchard	Findlay, Ohio	2.5	343	0.0073
Maumee	Antwerp, Ohio	87	2,049	0.0425
Maumee	Defiance, Ohio	90	5,530	0.0163
Maumee	Waterville, Ohio	112	6,314	0.0177
St. Joseph	Ft. Wayne, Ind.	57	1,060	0.0538
St. Marys	Ft. Wayne, Ind.	10	753	0.0133
Tiffin	Stryker, Ohio	13	444	0.0293

[1] Minimum seven-consecutive-day average flow which probably will be equaled or exceeded in 80 per cent of the years.

It should be emphasized that maximum demands for irrigation use will occur during the months of lowest stream flows.

Although there may be different views on the consumptive water use for irrigation in humid areas the following assumptions may be considered to demonstrate the effect on stream flow. An irrigation rate of one inch per week applied in 12 hr. will be representative of peak rates for a given field. Distribution is such that one-seventh of the total area is being irrigated at one time and the total seasonal need is five inches. The consumptive use is approximately equal to the total pumpage.

On the basis of the rate of water use per acre and distribution of water demand with respect to time, the required flow per acre irrigated would be .012 c.f.s. The total critical flow in the Maumee River at Waterville (120 c.f.s.) would be needed to serve 9,300 acres, which is approximately 0.23 per cent of the area above Waterville. Similar computations for other areas and streams will show that only a minor portion of the agricultural area can be irrigated from natural stream flow during critical flow pe-

riods even if the total flow is used. In many instances there would be serious damaging effects if consumptive uses for irrigation amounted to a significant part of the total low flow.

Any substantial development for the use of surface waters for irrigation in this or similar areas would have important conflicts with other water uses unless such development is accompanied by storage of surplus water during periods of high runoff. It can be readily appreciated that sewage treatment requirements, determined with regard to existing low flow conditions, would be radically changed by the use of natural stream flow for irrigation purposes.

Conclusion

In the development of irrigation water use in the eastern United States the apparent trend is toward development little by little on an individual basis. Initial individual irrigation uses, particularly where water is withdrawn from larger streams, may be developed with little thought or recognition of the serious conflicts which would occur with an accumulation of such uses. Advantages of supplemental irrigation by initial installations in an area have encouraged others in the area to make similar investments. Frequently the result was insufficient water for the total irrigation demand, together with reduced critical flows downstream. The probable effects on stream flow have reached the proportions in some areas in the East to cause concern by U. S. Geological Survey personnel as to the interpretation of stream flow data unless detailed investigation is made of irrigation water uses (10).

It is predictable that, sooner or later, there will be comprehensive legislation for control of water use in the East due to growing competive uses of water. This control will result from either advanced thinking and planned legislation or from necessity based on court action brought about by developing conflicts. In either case the attention of those responsible for water pollution control programs, as well as those interested in other water uses, including public and industrial water supplies and agricultural uses, should be directed toward control which will best serve the public interest. It should be kept clearly in mind, also, that the problems are likely to be interstate and regional in character.

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DISTRIBUTION OF COLIFORM BACTERIA AND OTHER POLLUTANTS IN TIDAL ESTUARIES *

By Bostwick H. Ketchum

Woods Hole Oceanographic Institution, Woods Hole, Mass.

In an estuary, any pollutant which is dissolved or freely suspended in the water is distributed by the circulation both upstream and downstream from the outfall location. A *conservative* pollutant, one which does not change with time, is distributed in a way that can be predicted from the salinity of the estuary. Many pollutants, including coliform bacteria and the biological and chemical oxygen demand, decrease with time in the water of the estuary, and this change is superimposed on the effects of the circulation. The distribution of these "time variable" or non-conservative pollutants can be derived if both the rate of the circulation and the independent rate of decay can be evaluated.

This paper attempts to explain the prediction of the distribution of pollution in estuaries. The prediction of the distribution of a non-conservative pollutant offers many difficulties which are as yet incompletely solved. The state of knowledge and ability to predict the distribution of coliform bacteria will be presented as an example of techniques available at present.

Distribution of Conservative Pollutants

When a conservative pollutant is introduced into an estuary at a constant rate, it will eventually achieve a steady-state distribution up and down the estuary which depends upon the rate of introduction and the rate of the circulation. Although the rate of circulation in estuaries may differ widely, the distribution of salt and fresh water is a direct indication of circulatory processes. The steady-state distribution of a conservative pollutant is directly related to the distribution of fresh and salt water, and can be derived from the results of a salinity survey of the estuary.

It should be pointed out that the conditions which will be discussed are average conditions and local concentrations will certainly differ. By the time the steady-state distribution is achieved, however, these local variations will be at a minimum. It is obvious the average concentrations of a pollutant in a total cross section will be much less than the local concentration immediately over the outfall. For the distribution immediately adjacent to the outfall the investigations of Rawn and Palmer (1) will be found a useful adjunct to the method described in this paper. The local distribution of pollutants will also be modified by different wind conditions. Local eddies and areas of slower than average circulation may be expected near headlands and in restricted basins. No effective general procedure for these local conditions has been developed, and a knowledge of the individual conditions and characteristics of the estuary is essential for the wise application of the methods described.

* Contribution No. 772 from the Woods Hole Oceanographic Institution, Woods Hole, Mass. Presented at 1955 Meeting, Texas Water and Sewage Works Assn.; College Station, Texas; Mar. 6–11, 1955.

Average Concentration of Pollutant

At the location of the outfall the pollutant will become mixed and its average concentration (C_o), achieved when steady-state conditions are reached, is given by:

$$C_o = \frac{P}{R} f_o \qquad (1)$$

in which P is the rate of supply of the pollutant, R is the rate of river flow and f_o is the average fraction of fresh water in the complete cross section of the estuary which passes through the outfall location. This is calculated from the average salinity (S_o) at that location by:

$$f_o = \frac{S_b - S_o}{S_b} \qquad (2)$$

in which S_b is the salinity of the source sea water in the mixture (2).

Downstream Distribution

In the steady-state, a conservative pollutant must pass seaward through every complete cross section at the same rate it is being introduced at the outfall. This same criterion of the steady-state condition applies to the fresh water, and the distribution of the pollutant downstream from the outfall is directly proportional to the distribution of river water. At any downstream location, the average concentration in the cross section, C_y, will be given by the relationship:

$$C_y = C_o \frac{f_y}{f_o} = \frac{P}{R} f_y \qquad (3)$$

in which C_y and f_y are the average concentration of pollutant and river water in the complete cross section (y).

Upstream Concentration

The pollutant will also be carried upstream from the outfall by the mixing process and the flood and ebb of the tidal currents. Above the outfall there will be no net exchange across any boundary when the steady-state distribution is reached. The quantity carried upstream will be exactly balanced by the quantity carried downstream. This is the criterion of the steady-state condition that applies to the salt content of the water. The upstream distribution of a conservative pollutant will be directly proportional to the distribution of salinity, as given by:

$$C_y = C_o \frac{S_y}{S_o} \qquad (4)$$

It is clear from these relationships that a knowledge of the distribution of salinity is essential in order to predict the expected distribution of pollutants. The salinity distribution in an estuary will be different at different times, depending largely upon fluctuations in river flow. However, an increase in river flow will produce an increase in the fraction of fresh water throughout the estuary. If there is a direct proportional relationship between river flow and the fraction of fresh water, the pollutant concentration will be independent of river flow, and will depend only upon the rate of supply. Its concentration may, therefore, be more nearly constant than the concentration of river water in the estuary.

Illustration

The foregoing relationships make it possible to predict the expected distribution of a conservative pollutant in an estuary. As an illustration, the expected distribution of a pollutant has been derived from the distribution of salt and fresh water in the Raritan River and Bay. An observed distribution of fresh water (3) is shown in Figure 1. Four locations have been selected arbitrarily for the position of an outfall, and the expected pollutant concentration from each is expressed as the percentage of the concentration obtained if the pollutant were uniformly mixed in the river water $\left(\frac{P}{R} = 1\right)$. Downstream from the out-

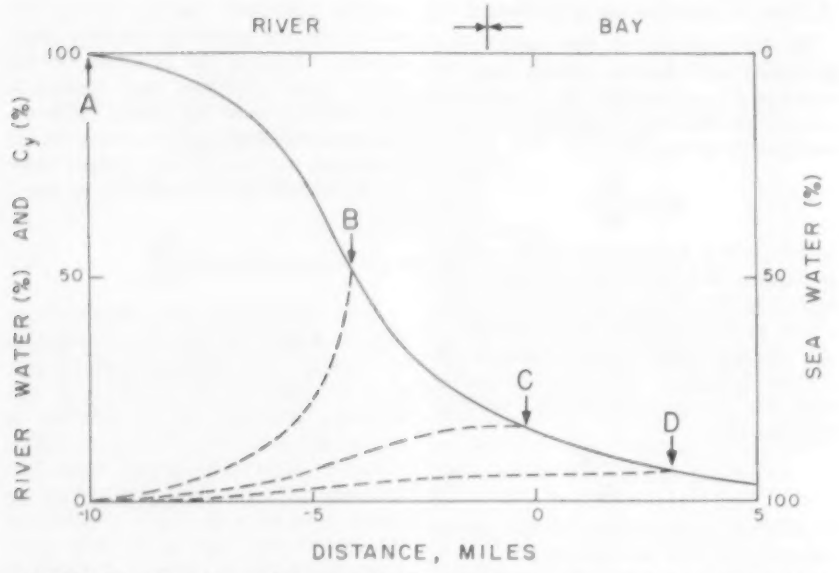

FIGURE 1.—The observed average distribution of river water, and the calculated distribution of a conservative pollutant (C_y) in Raritan River and Bay for four outfall locations.

fall the pollutant concentration is the same as that shown by the solid line, which describes the distribution of fresh water. Upstream from the outfall the dashed lines show the expected concentrations. Two important results of moving the outfall downstream are:

1. The concentration of pollutants upstream from the outfall is decreased.
2. The concentration of pollutants downstream from the outfall is unchanged.

Distribution of "Time Variable" Pollutants

Pollutants which decay or decrease with time will be less concentrated everywhere in the estuary than the predictions for the conservative pollutants. The change with time is superimposed on the circulation processes, and the joint and simultaneous effects of both determine the ultimate distribution. If the rate of supply is constant, a steady-state distribution will be attained also.

The mortality of coliform bacteria has been studied under laboratory conditions (4) (5) and a description given of the bactericidal effect of sea water which is found at all times of the year. The population change due to mortality alone is:

$$C_t = C_o e^{kt} \qquad (5)$$

in which C_o and C_t are the size of the population originally and at time t; e is the base of the natural logarithms, and k is the coefficient of mortality which is negative so that e^{kt} is less than unity. The laboratory experiments indicate the population of coliform bacteria decreases to a one-tenth its original size in a period of time ranging from one-half day to two or three days. Since the flushing time of most estuaries is longer than this, the bacterial mortality can be a most important factor in the distribution.

Practically all the decrease of coli-

form bacteria in the Raritan River can be accounted for by the circulation, the bactericidal action of sea water and the rate of consumption of the bacteria by zooplankton organisms in the water (6). The following relationship was used for this computation:

$$C_n = (C_o)_n \frac{r_n}{1 - (1 - r_n)e^k} \quad (6)$$

in which n is a tidal excursion segment as defined by Ketchum (7) and r_n is the exchange ratio of that segment. The increment to the population in the segment considered $[(C_o)_n]$ from the adjacent segment $(n - 1)$ in the direction of the source of pollution, is given by:

$$(C_o)_n = C_{n-1} \frac{f_n}{f_{n-1}} \quad (7)$$

Since the populations in the segments closer to the outfall $(n - 1, n - 2$, etc.) can also be defined by Eq. 6, successive substitutions relate the population in any segment to the average population at the outfall. At this location the rate of supply is given by Eq. 1 depleted by mortality while within the segment, *i.e.*, multiplied by the fraction $\left[\left(\frac{r_n}{1 - (1 - r_n)e^k}\right)\right]$. If the mortality coefficients and the exchange ratios are equal for all segments the relationship can be reduced to:

$$C_n = C_o \frac{f_n}{f_o} \left(\frac{r}{1 - (1 - r)e^k}\right)^n \quad (8)$$

Similar reasoning indicates that the population upstream from the outfall can be given by:

$$C_n = C_o \frac{S_n}{S_o} \left(\frac{r}{1 - (1 - r)e^k}\right)^n \quad (9)$$

In both Eqs. (8) and (9) the exponent (n) is the number of segments from the outfall to the locality considered, including the segment containing the outfall.

The effect of mortality can be di-

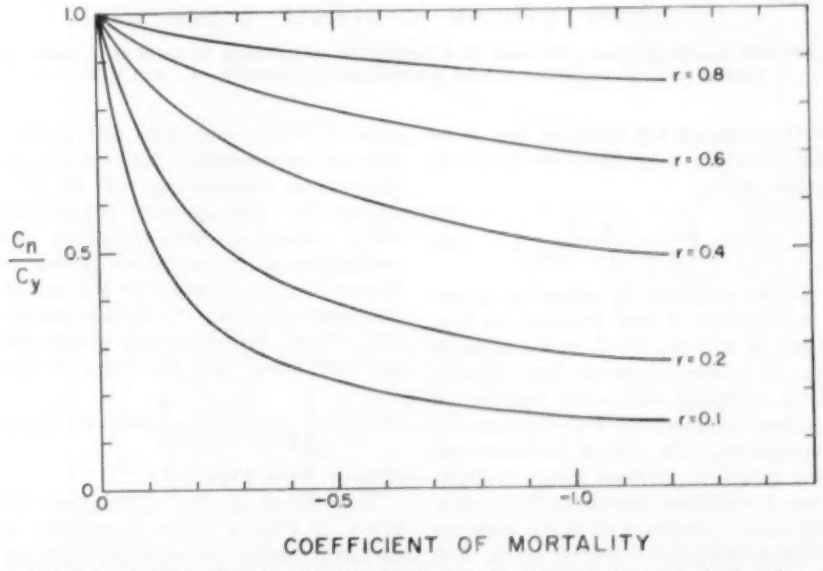

FIGURE 2.—Relative size (C_n/C_y) of a population of coliform bacteria, at the location of the outfall, produced by various circulation rates (r) and mortality coefficients.

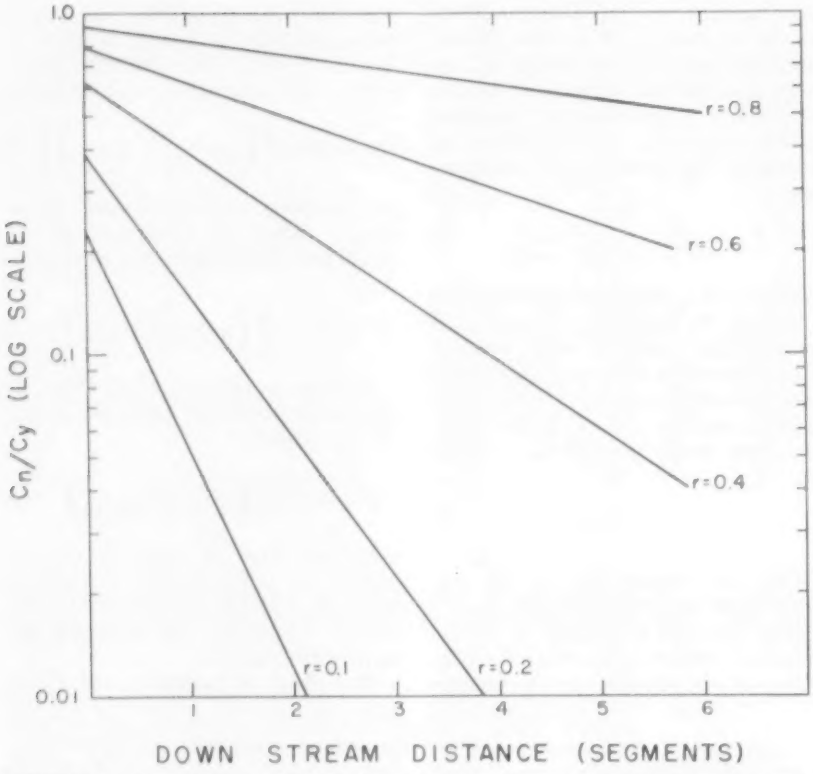

FIGURE 3.—Downstream decrease of a population of coliform bacteria as a result of various rates of circulation (r) for a coefficient of mortality of -0.51 tides^{-1}.

rectly assessed by dividing Eq. 8 by Eq. 3, or Eq. 9 by Eq. 4 so that both reduce to:

$$\frac{C_n}{C_y} = \left(\frac{r}{1 - (1 - r)e^k} \right)^n \quad (10)$$

If the coefficient of mortality is zero ($e^k = 1$) Eqs. 8 and 9 reduce to Eqs. 3 and 4, and Eq. 10 gives C_n equal to C_y. It is also apparent that for any given exchange ratio, any increase in the rate of mortality will decrease the population. The larger exchange ratios, however, produce larger populations for a given mortality rate, since the water remains within the segment of the estuary for a shorter time. For example the average age of the river water in a segment with an exchange ratio of 0.8 is only 1.25 tidal cycles, whereas an exchange ratio of 0.1 indicates an average age of 10 tidal cycles (3). The expected populations for a variety of exchange ratios and coefficients of mortality are shown in Figure 2. The location of the outfall has been used ($n = 1$) so that successive effects of downstream circulation are eliminated and the value of the fraction $\left[\frac{r}{1 - (1 - r)e^k} \right]$, may be found directly from Figure 2.

The effects of the circulation are given in Figure 3 for a variety of exchange ratios (r) and a single coefficient of mortality ($k = -0.51$, $e^k = 0.6$). The more rapid the circulation

the larger the relative population size at a distance from the outfall. This again, reflects the brief period of time required for the water to move through the successive segments. The slower circulations, however, permit a considerable population decrease as a result of mortality, and the resultant populations are much smaller than they would be if decreased by circulation alone.

The joint effects of the circulation of the estuary and the mortality of a coliform bacteria population have been computed for three possible outfall locations in Raritan River and Bay. The coefficient of mortality used was the same as previously observed (6). The exchange ratio for this estuary is nearly constant from segment III to segment X, a distance of nearly 10 mi. where it ranges from 0.285 to 0.397 (7), with an average value of 0.339. The upper curve of Figure 4 repeats the solid curve of Figure 1 and gives the population expected from dilution alone (C_y) with no mortality of the bacteria for an outfall at location *A*. The lower curves show the effects of mortality by indicating the fraction of the population (C_n/C_y) expected when the coefficient of mortality is -0.578 tides^{-1}. Multiplication of the value in the upper curve by the fraction at the corresponding location in the lower curve gives the population to be expected. These are relative to a population size of unity for the pollution bacteria mixed in the river water with no mortality. For example the maximum relative populations at the outfall locations would be 0.538 for *A*, 0.278 for

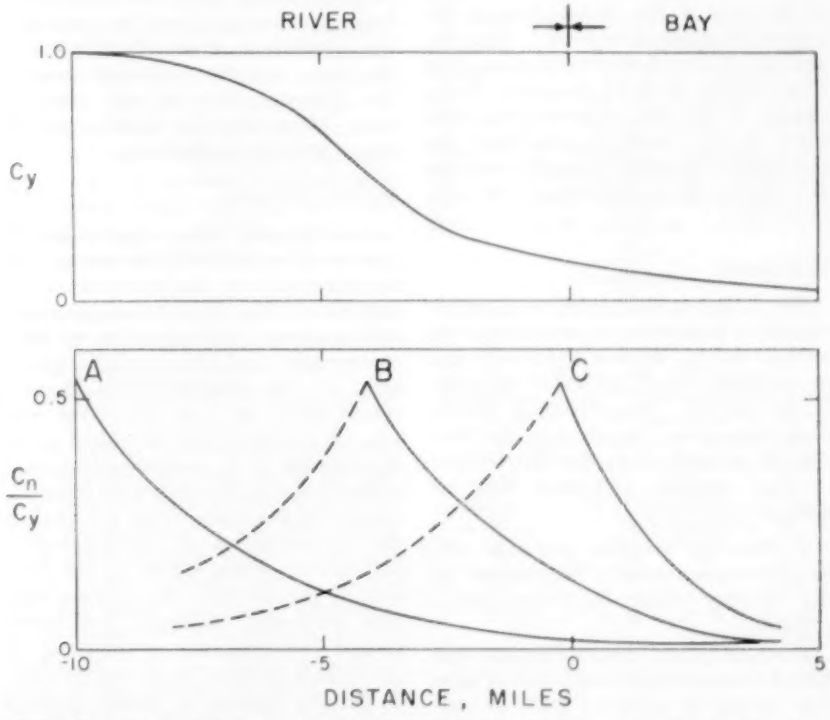

FIGURE 4.—The relative size (C_n/C_y) of a population of coliform bacteria calculated for three possible outfall locations in Raritan River and Bay.

B and 0.092 for *C*. The peak concentration, therefore, is decreased by downstream placement of the outfall, but to a greater extent than can be expected for a conservative pollutant.

Illustration

A discussion of the distribution of bacteria from an outfall at location *B* will clarify the effects of different locations on the distribution of time variable pollutants in general. At outfall location *B*, the population is greater than it would be if the outfall were located either upstream or downstream from *B*. The intersection with the curves for other locations (Figure 4) shows the extent of this greater population size. For an outfall at *B*, a stretch of the river nearly five miles in length would have a higher population of coliform bacteria then could be expected if the outfall were to be located at either *A* or at *C*. Also, the population would be greater than that from outfall *A* at all locations downstream from the intersection with curve A, but would be less than the population of outfall *C* at all locations downstream from the intersection with curve *C*.

Conclusion

For a time variable pollutant, therefore, it is clear that no location for an outfall can be selected which will improve conditions at all other locations in the estuary. The following conclusions concerning the effect of the location of the outfall on the distribution of time variable pollutants may be drawn:

1. The time variable pollutant will be less concentrated everywhere in the estuary than the conservative pollutant.

2. At and near the outfall, the pollutant concentration will be greater than would be obtained from an outfall located either upstream or downstream.

3. Like the conservative pollutant, the upstream concentration can be reduced by a downstream displacement of the outfall.

4. In contrast to the conservative pollutant, the downstream concentration can be reduced by an upstream displacement of the outfall.

Discussion

The methods described in this paper predict the distribution of both conservative and time variable pollutants in an estuary from the results of a salinity survey. The relative merits of different locations for an outfall within an estuary, therefore, can be evaluated. No assumptions concerning the mixing process are involved for the conservative pollutant, though the results are expressed in terms of average conditions, and local variations from the average may be expected. The prediction of the distribution of the time variable pollutant involves the determination of an exchange ratio, which may be based upon assumed mixing conditions.

Outfall Location

Some general rules concerning the location of an outfall in an estuary can be proposed. A displacement of the outfall location downstream decreases the upstream concentration of both conservative and time variable pollutants. If the objective is to improve conditions in the river end of the estuary, the outfall should be placed as far downstream as is economically practicable to obtain the maximum improvement. Downstream from the outfall, the concentration of a conservative pollutant is independent of the location of the outfall. For a time variable pollutant, however, the downstream concentration is increased as the outfall location is moved seaward. If bathing beaches or fishing grounds near the mouth of the estuary are important considerations, an upstream

location of the outfall would be indicated. A precise evaluation of the conditions which it is desired to improve is necessary before the selection of an outfall location in an estuary is possible.

Stommel (8) has described a method for predicting the distribution of pollutants in a vertically mixed estuary. This depends upon the calculation of the average mixing conditions from the distribution of fresh water, and the determination of the expected pollutant distribution by relaxation methods. Relationships, similar to those described above, between the distribution of pollutants and the fresh and salt water downstream and upstream from the outfall were independently derived by Stommel.

Exchange Ratio

The prediction of the distribution of a pollutant which decreases with time requires the determination of an exchange ratio for each tidal excursion segment. This ratio is defined as the fraction of the water in a segment which is lost downstream with each tidal cycle. A method for computing an exchange ratio from the intertidal volume and the high tide volume of the various segments has been described by Ketchum (3, 7). This depends upon the assumption that the water is uniformly mixed at high tide within a segment defined in length by the excursion of water on the flooding tide.

This method was adequate for estuaries as widely different as the Raritan River, and the Bay of Fundy (7) (9). With modifications it was satisfactory for a small estuary, Great Pond, and for a fjord, Alberni Inlet. Austin (10) has found reasonable agreement in the Mobile River and the inner end of Mobile Bay, but found errors of about 25 per cent in the outer end of Mobile Bay. Stommel (8), however, found that this method could not be used to describe the salinity distribution in the Severn Estuary.

The calculation of exchange ratios from intertidal and high tide volumes within the estuary should not be used without adequate salinity data to verify that the assumption concerning mixing is a reasonable approximation. However, if adequate salinity data are available the exchange ratios can be determined directly from the accumulation of fresh water, since:

$$r_n = \frac{R}{Q_n} \qquad (11)$$

in which Q_n is the total volume of river water accumulated in the tidal excursion segment (2) (9) (11). This calculation assumes steady-state conditions, but makes no assumptions concerning the mixing process. A valid exchange ratio is obtained provided river flow has been constant, so that the steady-state distribution has been attained. It is recommended that enough salinity data be collected in order to confirm the exchange ratio derived from tidal volumes, or to derive the exchange ratio directly from the accumulation of river water, before the results are used to predict the distribution of pollution.

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CORRECTION

The abstract "Investigations on the Dynamics of Aeration—Effects of Some Surface Contaminants" (THIS JOURNAL, **27**, 10, 1215; Oct., 1955) was not complete. The complete abstract is:

Investigations on the Dynamics of Aeration—Effects of Some Surface Contaminants. BY A. HOLROYD AND H. B. PARKER. *Jour. Inst. Sew. Purif.* (Brit.), 280 (1952).

A laboratory study of the effects of five selected surface-active agents (Turkey Red oil, Commercial Teepol, Commercial Lissapol-N, sodium oleate, and bentonite) on the dynamics of aeration led to the following conclusions:

1. The rate of transfer of oxygen through an aeration surface is greatly influenced by the fluid flow conditions in the water adjacent to that surface. Estimations of the transfer rate, f, have been made for diffused air bubble surface, heaving water surface, and certain stagnant film surfaces including still water surface.

2. With addition of certain contaminants, the changes in f vary considerably with the character of the aeration surface.

3. Teepol, Lissapol and Turkey Red oil had the most important observable effects on aeration. Sodium oleate proved too reactive and unstable under the conditions of the experiments for reliable measurements, but indications are that soaps also would behave towards aeration like the more stable surface-active agents.

4. Up to 100 p.p.m. in clean water, the surface-active agents showed effects previously observed when sterilized sewage was aerated. Sewage increased the effects above those noted with surface-active agents in pure solution.

5. The most significant reduction in surface transfer rate by surface-active agents was found at the diffused air bubble surface. The bubble surface, because of a barrier layer of adsorbed surfactant, must be considered a form of stagnant film aeration surface.

6. Less severe depressions of the surface transfer rate due to surfactants were observed at stagnant film surfaces of relatively low f value.

7. Heaving water surface had the highest clean water surface transfer rate and suffered no appreciable depression from this high level in the presence of surfactants.

8. Still water aeration rates present special problems in their determination, but these rates seem to suffer no appreciable depression in the presence of surfactants.

9. The behavior of surfactants towards aeration exhibits certain individual variations with increasing concentration.

10. Bentonite, a highly colloidal clay, had no influence on transfer rates at any of the aeration surfaces examined.

H. P. O.

THE B.O.D. TEST AND TOTAL LOAD*

By A. M. Buswell

Chief, Illinois State Water Survey, Urbana, Ill.

In a previous paper (1), it was shown that the total load imposed on a stream by a waste could not be calculated from the rate of oxygen uptake as observed in the B.O.D. test or the 5-day B.O.D. value. The rate of oxygen absorption was shown to be dependent on the number of bacteria available to carry out the biochemical oxidation and not on the amount of oxidizable material present. These studies were made using ammonia as the oxidizable material. Almost simultaneously, Hoover (2) reported similar results using carbonaceous matter. This conclusion was not entirely unexpected and in fact had been indirectly predicted by Phelps (3) who was the first investigator to make a mathematical study of the B.O.D. reaction. Acceptance of this fact marks the end of an era and the ideas of what constitutes the total pollution load must now be revised.

Considerations

Recent work has clarified thinking concerning the B.O.D. test and its implications. It has emphasized the fact that there are three kinds of B.O.D.:

1. Carbonaceous organic matter oxidized to water and carbon dioxide.
2. Ammonia oxidized to nitrite.
3. Nitrite oxidized to nitrate.

The carbonaceous organic matter is the most objectionable since it causes:

1. Depletion of oxygen below fish tolerances.
2. Production of nuisance by reduction of sulfates.
3. High chlorine demand.

The nitrogenous B.O.D., resulting from the oxidation of ammonia or nitrites, is less serious. Although it may cause oxygen depletion below fish tolerances, it would not produce nuisance nor cause high chlorine demand except in the case of breakpoint chlorination practice. Thus, it appears probable that in the future three B.O.D.'s instead of one may have to be determined.

The chemical oxygen demand (C.O.D.) was used as a measure of pollution nearly a century ago, with permanganate as the oxidizing agent. Since that time practically every chemical oxidizing agent has been tried and the literature is full of appropriate references.

In the opinion of the author all the chemical methods have merit. The fact that they give results varying among themselves and with the B.O.D. as well as varying from the stoichiometrically calculated values, offers opportunity for correlating the various results with the specific chemical nature of the pollutant. Such a correlation will be greatly aided by the Van Slyke method (4) as modified, which appears to measure accurately the total oxidizable carbon.

In addition to the three B.O.D.'s mentioned previously, it may well be worthwhile to determine several C.O.D.'s using the Van Slyke method as a reference. From the various B.O.D.'s and C.O.D.'s it may be possible to arrive at a conception of the kind as well as the amount of the total load. This involves considerable laboratory work, but it will cost only a small fraction of the capital expenditure involved in a sewage treatment works.

* Presented at 1955 Annual Meeting, Central States Sewage and Industrial Wastes Assn.; Rochester, Minn.; June 22–24, 1955.

However, the laboratory analytical data at best will be only a partial answer to the problems confronting the engineer designing a treatment plant. An oxygen load indicated in the laboratory report may or may not be a load on the particular stream into which the treated waste is to be discharged. For example, the ammonia B.O.D. will constitute a load only when the stream carries an adequate inoculum of nitrosomonas.

Data Needed

First, because it is simple, would be the determination of the chemical conditions existing in the river as compared with those found favorable to nitrification. In a laboratory medium, the optimum conditions were found to be 5 p.p.m. ammonia nitrogen, a pH of 8 to 8.5 and a temperature of 32° C. If the composition of the stream even approaches these conditions, ammonia may become a B.O.D. load.

Secondly, it will be necessary to determine whether there are sufficient nitrosomonas organisms in the stream to produce a 5-day B.O.D. from ammonia. This will require some special bacteriological techniques.

The minimum numbers of nitrosomonas organisms per milliliter required to remove 6 p.p.m. dissolved oxygen in 5 days in the presence of ammonia nitrogen are:

Temperature	Nitrosomonas Organisms per ml.
10° C.	240,000
15° C.	37,400
20° C.	5,310
25° C.	227
30° C.	25

The extent to which the rate constants on which these numbers are based would be altered by river conditions as compared to the laboratory conditions is unknown. It seems likely that a less favorable environment would be presented in the river. Therefore, the critical numbers would probably be larger than those shown.

Directions for determining the number of nitrosomonas organisms have been published elsewhere (5).

Conclusion

In view of the above discussion it appears that to obtain the best estimate of the total pollution load of a waste, the following determinations are needed:

1. The conventional B.O.D.
2. The Van Slyke combustion (total carbon).
3. One or more C.O.D. determinations.
4. Ammonia nitrogen.
5. Nitrosomonas count.

Careful interpretation of these data will give a much clearer picture of what will happen when a given waste is discharged into a stream than can be had from the limited parameters now used. They will certainly serve as a much sounder basis for the design and operation of sewage and industrial wastes treatment plants.

Acknowledgment

The work described herein was performed under a research grant from the Division of Research Grants and Fellowships, National Institutes of Health, U. S. Public Health Service.

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STATISTICAL ANALYSIS OF COLIFORM DATA

A DISCUSSION

By Richard Pomeroy

Consulting Sanitary Engineer, Pasadena, Calif.

An important point has been made by Thomas (1) in showing that the geometric mean of a set of bacterial data is not an adequate measure of the seriousness of pollution at the sampling point. It is important that proper weight be given to the occasional high values. There are two points in his article which deserve comment.

Plotting on Probability Paper

In the method of plotting used, the plotted probability is obtained by dividing the accumulated number of results by $N + 1$ instead of by N, N being the number of results. The usual argument for this practice is that the last point otherwise cannot be plotted. This technique is not consistent with an assumption that the results are distributed according to the normal curve of errors.

Suppose that a series of 50 results is plotted. A point that actually represents 98 per cent of the results will be calculated and plotted at 96.1 per cent, and other points will be shifted similarly. This amounts to a modification of the probability paper. There is no reason why a modified paper should not be used if it gives good correlations. Thus this procedure could be satisfactory provided the modification is consistent. But the modification produced by this method of plotting will vary with the value of N. Thus, if there are 100 results instead of 50, the point which actually corresponds to 98 per cent of the results will not be plotted at 96.1 per cent but at 97 per cent. For a large set of data the plot approaches that which would be obtained by dividing by N instead of $N + 1$.

A theoretically correct way to make a plot on probability paper is to plot for each result a horizontal line representing the interval of percentages from a total without that point to the total with that point. Thus, if there are 50 results, the first represents the interval from 0 to 2 per cent, the second is from 2 to 4 per cent and the last is from 98 to 100 per cent.

TABLE I.—Empirical Results of 60 Eight-Penny Tosses

Number of Heads	Number of Tosses
0	0
1	2
2	9
3	12
4	25
5	5
6	5
7	2
8	0

The plot then is a series of horizontal lines. The ends of these lines may be joined by vertical lines, making a staircase figure.

Table I shows the results of a set of 60 tosses of eight pennies. A staircase type of plot of these results is shown in Figure 1. At the bottom is a horizontal line extending infinitely to the left toward zero, and at the top is a line extending infinitely to the right toward 100 per cent. The plot is reversible, as it should be. It is reversible in the sense that the results can be tabulated as decreasing number of tails instead of increasing, with a resulting plot that is the exact mirror image of the plot in Figure 1.

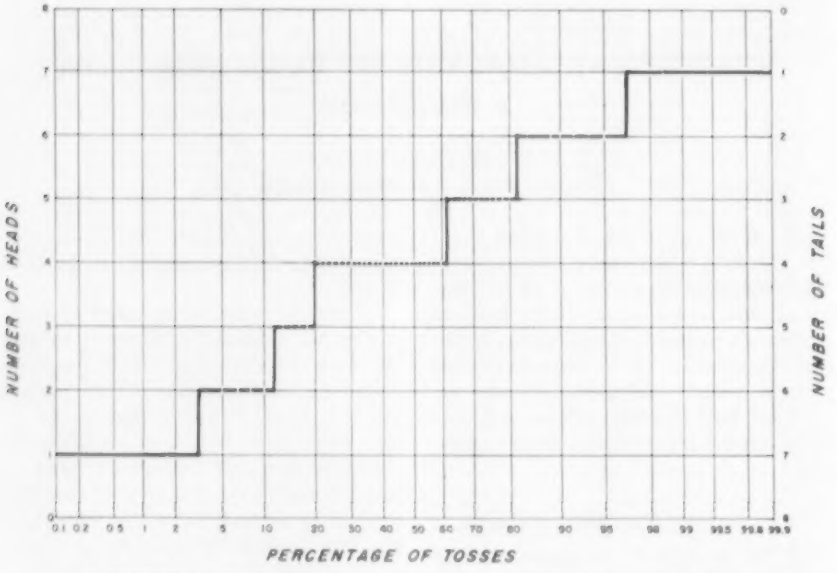

FIGURE 1.—Probability paper plot of a set of 60 eight-penny tosses.

It may be objected that the extension of the line representing the highest value of the variable infinitely to the right is inconsistent with the fact that the variable can have higher values. Generally it can, but all that a given set of results can show empirically is the highest value found. In the example of tossing pennies used here, a line approximating the slope of the stairway might be extended beyond the highest observed result, which was seven tails. But this extension, if representing an infinite set of eight-penny tosses, would not be a straight line, and could not go higher than eight tails. To a greater or lesser extent this is typical of nearly all real sets of data.

For sets of less than 100 results, the conclusions may be substantially affected by the way that the plotting is done. Thus the results for contact tank effluent in Figure 1 of the article by Thomas seem to show a geometric standard deviation of $\sigma_g = 6$, with a geometric mean of 11 and an arithmetic mean of 42. If the data are plotted in accord with the assumption of true conformance to the probability curve, the value of σ_g is about 3.4. The geometric mean is 17 and the arithmetic mean is 29. This latter figure receives support from a direct calculation of the arthimetic average of the points insofar as they can be read from the curve. This average appears to be about 30 (an average result of 10 is assumed for the 43 indeterminate low results).

In a technique recently published (2) for statistical treatment of bacterial data, the method amounts to the locating of points at the midpoints of the lifts of what otherwise would be the staircase type of plot. This provides a quick way to indicate by a few points the course of the distribution line or curve.

Practicability of the Arithmetic Mean

It is desirable, as pointed out by Thomas, that coliform standards be written to give weight to the high values. This can be done by limiting the high percentiles or by limiting

the function expressed by Thomas as the arithmetic mean.

Some caution should be exercised in the interpretation of the formula for the artithmetic mean. Validity of the derivation by Thomas (1) depends on the assumption that the logarithms of the M.P.N.'s follow the normal distribution, which is never more than an approximation. When σ is large, a high arithmetic mean is calculated which depends almost entirely upon high values which are usually in an extrapolated range. For example, in Figure 1 of the previously mentioned paper by Thomas, the actual arithmetic mean of the upper curve, if it could be known, would depend almost entirely upon the "four values 110,000 or greater." It is dubious practice to extend a line on probability paper, or the corresponding equation, to a range substantially beyond the data.

An attempt was made to use arithmetic means of M.P.N.'s of beach waters, with sets of data having high σ values. It was found that if use were made of a straight line on the probability plot to try to calculate the mean, this average would depend in substantial part on impossibly high values.

If the arithmetic average is specified in a water quality standard, consideration must be given to the fact that a set of bacterial data may show up as a curve when plotted on log probability paper, instead of a straight line. This would be the case in a stream that normally shows little pollution but which occasionally receives a large flow of sewage. It is also true for the data on settling tank effluent in Figure 1 of the paper by Thomas.

Number of Dilutions

The subject of numbers of tubes and numbers of dilutions deserves some consideration in connection with this discussion.

Many sets of bacterial data are obtained by setting three, four or five tubes in each dilution, but often using only three dilutions. Wherever there is much variability of pollution, use of only three dilutions will leave many indeterminate samples at the ends, all minus or all plus. When the purpose of the test seems to be to determine only whether a coli-aerogenes count is above or below a certain level, it might appear that this is adequate. The paper discussed makes it clear that more information is needed. In most cases it would be of great value to know how high the very high values are, and it would also be of interest to know the distribution of the low values.

It would be better generally, therefore, to set more dilutions. Under most conditions a pattern of five tubes in each of three dilutions can be changed advantageously to three tubes in each of five dilutions, or even to two tubes in each of five or six dilutions.

The apparent greater precision of the test when five tubes are used in three dilutions is somewhat illusory. The larger number of tubes gives some increase of precision if the split is principally in the middle group. But if most of the last group are positive, the result may be no more accurate than if only two or three tubes were used in that dilution, with three or two tubes in a further dilution. A similar condition exists at the other end of the series.

The author (Thomas) of the paper is to be congratulated for an important contribution to the interpretation of bacterial data. In spite of the uncertainty about the meaning of the function which is presented as the arithmetic mean, it is not unreasonable that this function may be used for quality criteria, at least until a more acceptable proposal is made.

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PHILOSOPHY OF WATER POLLUTION CONTROL IN CALIFORNIA *

By A M Rawn and Vinton W. Bacon

Respectively, Chairman and Executive Officer, California State Water Pollution Control Board, Sacramento, Calif.

The California water pollution control program has developed and to some extent crystallized many control practices since its initiation in 1950. The subject is considered here in three parts: (a) brief description of administrative organization; (b) discussion on the basic concepts of water pollution control legislation; and (c) statement on how the water pollution control program enters into the important proposals for a state department of water and the California Water Plan.

Administrative Organization

In 1949, the California Legislature concluded that water pollution control was primarily a matter of regional concern, and in enacting the Dickey Water Pollution Control Plan into law, the Legislature created a decentralized state organization by dividing the state into nine regions as shown in Figure 1. The regional boundaries, for the most part, are based on major watershed areas.

Within each of these regions, there is appointed by the governor a five-man regional water pollution control board. Each regional board member must be selected from a specified field: water supply, irrigated agriculture, county government, city government, and industrial waste. There are no ex-officio state officials on the regional boards. A 14-man State Water Pollution Control Board was also created by the 1949 legislation. Nine board members are appointed by the governor from seven designated fields: water supply, irrigated agriculture, county and city governments, industrial waste, industrial water use, and public sewage disposal. The remaining five members are state officials representing the Departments of Agriculture, Fish and Game, Natural Resources, Public Health, and the Division of Water Resources. Twelve major fields of interest are represented on this board. Board members all serve without compensation. A comparison of the board membership is as follows:

Fields Represented on Water Pollution Control Boards

Regional Board (5 members)	State Board (14 members)
Water supply	Water supply
Irrigated agriculture	Irrigated agriculture
County government	County government
City government	City government
Industrial waste	Industrial waste
	Industrial water use
	Public sewage disposal
	Dept. Agriculture
	Dept. Fish and Game
	Dept. Natural Resources
	Dept. Public Health
	Div. Water Resources

As regulatory agencies, the regional boards have three principal duties: (a) formulating and adopting long-range plans and policies; (b) prescribing and enforcing waste discharge requirements; and (c) coordinating the interests of other agencies.

The State Board, serving in a coordinating and appellate capacity, is directed to: (a) formulate a statewide policy for control of water pollution with due regard to the authority

* Presented at 1955 Annual Meeting, California Sewage and Industrial Wastes Assn.; Riverside, Calif.; April 27–30, 1955.

FIGURE 1.—Water pollution control regions for the state of California. Boundaries in general are based on major watershed areas.

of the regional boards; (b) administer state-wide programs of research or of financial assistance for water pollution control; (c) allocate funds for the administrative expenses of the regional boards; and (d) review the acts of any regional board that fails to take or obtain appropriate action to correct a condition of pollution. In connection with the last duty, and to emphasize the effectiveness of the decentralized program, during the five years of operation the State Board has had neither to review nor intervene in a single case.

Basic Concepts

Two basic concepts in the California program are apparent in the administrative pattern, namely, decentralization and the balance of interests represented on the boards. Decentralization, in this case, means far more than the creation of district offices responsible to a central organization. In their pollution control activities,

the regional boards are autonomous. They have been assigned the primary responsibility of controlling water pollution within their respective regions. In this capacity they are basically local agencies governed by local people representing local interests.

The second fundamental concept, that of providing an equitable balance of interests in the consideration of economical waste disposal and preservation of the beneficial uses of water, is one of the strongest aspects of the water pollution control legislation. This necessary balance of the interests of the water user with those of the waste discharger is achieved through the broad and varied representation on both state and regional boards.

A third concept expressed in the water pollution law is that the control boards, in setting waste discharge requirements, may prescribe end results only. This provision eliminated the previous system of issuing permits and reviewing plans for sewerage works. Under the present laws, the control boards prescribe waste discharge requirements only in relation to conditions existing from time to time in the disposal area or receiving water. This does not mean that the boards are not interested in the design or operation of sewerage facilities. To the contrary, the staffs of these boards can, and do, informally advise the waste discharger on problems of design or operation. But, the law clearly states that the boards may not specify the design, location, type of construction, or particular manner of correcting pollution. Persons ordered to correct or prevent pollution may do so by any lawful means.

Another principle of the 1949 legislation was the creation of a single agency to coordinate the efforts and interests of other agencies. Following a three-year study of water pollution, the Dickey fact-finding committee which drafted the water pollution control law, referred to the many governmental agencies concerned with the problems of waste disposal and water quality as the "57 Varieties." Actually, the committee found that there are more than 57 agencies in California at the federal, state, or local level having some interest, direct or indirect, in water pollution. The California law makes it possible for the waste discharger to negotiate with only one state agency. The principle of "one report, coordination and one regulation" is illustrated in Figure 2.

As illustrated, a prospective discharger of sewage or industrial waste is required to file only one report with the appropriate regional board. This report consists of a simple, single-page form. The board forwards copies of the form and any other pertinent data to all the interested and affected parties, thus starting the process of coordination. Where technical facts are lacking, the board asks state or local agencies to make special investigations. Before adopting tentative regulations, the board invites comments and recommendations of the interested agencies. Informal or formal hearings may be held to develop

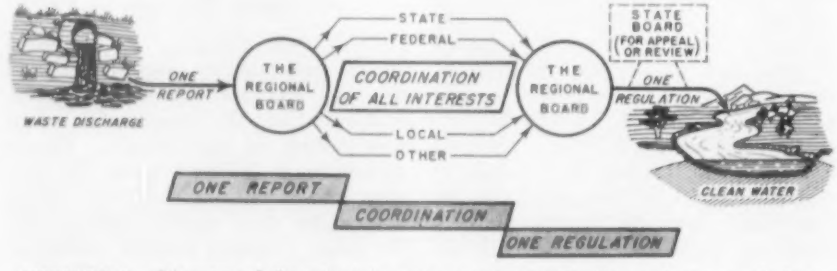

FIGURE 2.—Diagram of the principle—one report, coordination and one regulation.

additional facts. Finally, the board, in official meeting, prescribes the discharge regulations. The process is now completed, with the waste discharger receiving but one set of regulations.

Steps in Controlling Pollution

To demonstrate how the water pollution control boards protect the interests of other agencies, two of the four steps taken by the boards in preventing and abating pollution will be described. The four steps followed by a control board are:

1. Enunciation of the beneficial water uses which are intended to be protected.
2. Definition of water quality criteria to protect the beneficial water uses.
3. Prescription of waste discharge requirements.
4. Check on compliance and enforcement of requirements.

Although all four of these interrelated duties are important functions of the boards, the following comments will stress only the enunciation of beneficial water uses and the defining of water quality.

Enunciation of Beneficial Water Uses

When considering the first step, it is necessary to understand what is meant by beneficial water uses and why their enunciation becomes so significant in this program. In the California statutes "pollution" is defined as an impairment of the quality of water by sewage or industrial waste to a degree which adversely and unreasonably affects waters for domestic, industrial, agricultural, navigational, recreational, or other beneficial use. There is no pollution if beneficial uses of water are not *adversely* and *unreasonably* affected. Thus, it is evident that the first problem facing the control board, either in the development of long-range plans and policies or in the prescribing of requirements to regulate individual waste discharges, is that of determining the beneficial water uses to be protected.

The control board does not establish water uses. The California Water Pollution Control Act clearly states: "No provision of this division shall be construed as affecting the right of any person to the use of water for any beneficial use other than the use for disposal of sewage and industrial wastes." Water uses are established by factors such as the California Water Plan, authorized water projects, established and planned future water uses, water rights, the economy of the area, land uses, and local and regional planning programs.

Although the control board does not establish water uses, it does enunciate water uses which it intends to protect. Prior to making this declaration, the board considers established water uses, planned future uses, and the need for economical waste disposal. The board also recognizes that the total interest in any water pollution control program encompasses both economical waste disposal and preservation of the beneficial uses of water. To be certain that all interested parties are heard, the board invites comments on each case, holds informal meetings and discussions, conducts formal public hearings where necessary, and, lastly, after weighing the facts, acts upon the matter at a public meeting. In this way, the board serves as arbiter with the duty to hear all interested parties, to evaluate all factors, and then, to enunciate the water uses which it intends to protect.

Setting of Water Quality Criteria

After the control board has determined what water uses it will protect, the second step is to establish limits on physical, chemical and bacteriological characteristics or concentrations beyond which the quality of the waters are impaired for beneficial uses. The process of defining these limits, or "water quality criteria," can be

illustrated by a simple, hypothetical case.

Assume that a community, situated on the banks of an inland river, desires to dispose of its sewage and industrial wastes in the river. Let us further assume that, after considering all factors and interests, the control board determines it will preserve the river for irrigation, bathing and fishing. Therefore, the community will have to provide waste treatment to the extent that the river downstream from the outfall will meet water quality criteria suitable for these three defined uses.

In determining water quality criteria, the control board customarily consults with agencies and individuals who are experts in particular fields of water use. This consultation, although not mandatory, is the normal procedure. However, it should be emphasized that the final decision in all cases rests with the water pollution control board. In the hypothetical case, irrigation water authorities such as universities, agriculture experiment stations, the Agricultural Extension Service, the Division of Water Resources, the Department of Agriculture, etc., would be asked what would be a maximum concentration of dissolved minerals beyond which the river water would be considered impaired for irrigation of certain crops. Perhaps the authorities recommend a maximum of 700 p.p.m. of dissolved minerals in this case, and the control board adopts this value as one of its criteria. This figure should have a reasonable margin of safety in it, and therefore, should not be considered the point below which the board is unconcerned with the waste discharge or above which the board must rush into enforcement proceedings. When the dissolved minerals concentration is considerably less than 700 p.p.m., conditions in the river should be satisfactory. Should the concentration of dissolved minerals approach the 700 p.p.m. criterion, particularly over a prolonged period, the possibility of unreasonable impairment (pollution) would be recognized by the control board. In this case, the waste discharge would be subject to immediate study and careful checking to determine whether enforcement proceedings should be initiated against the discharger.

To complete the hypothetical case, criteria for irrigation water use would also be determined for characteristics such as the sodium ratio and boron concentration. Using the same procedures, additional criteria would be determined, after consultation with public health, fish and game, and other authorities, for protection of the bathing and fishing uses of the river water. Again, a reasonable margin of safety would be embodied in these criteria to assure protection to public health and fisheries resources as well as to the agricultural interests.

Table I shows the defined water quality criteria for each beneficial water use in the hypothetical case. The first column gives the three beneficial uses to be protected in the river water. The second column shows the criteria which have been prescribed by the control board. For irrigation use, the maximum limits are 700 p.p.m. dissolved minerals, 40 per cent sodium ratio, and 0.5 p.p.m. boron. For bathing use, 10 coliform bacteria per milliliter are considered as the maximum concentration. And for fishing use, the minimum limit of dissolved oxygen is defined as 5 p.p.m. This column has been labeled the "area of not unreasonable impairment." As long as the limits shown in this column are not exceeded, the quality of the river water is not unreasonably impaired for the uses indicated and there is no pollution. Regulations controlling waste discharges to the river would be prescribed so that conditions in the receiving water would be maintained within these limits.

Should the limits be exceeded, the

TABLE I.—Defined Water Quality Criteria for Hypothetical Case of Beneficial Water Use

	Area of Not Unreasonable Impairment	Area of Unreasonable Impairment	
	Satisfactory	Caution	Danger
Beneficial use to be preserved in river water	Water quality criteria prescribed by control board to prevent pollution		Possible conditions if pollution not controlled
Irrigation	700 p.p.m. dissolved minerals, max. 40 per cent sodium ratio, max. ½ p.p.m. boron, max.		High concentrations; crop damage and loss of production
Bathing	10 coliform bacteria per milliliter, max.		High coliform counts; an actual hazard to public health (contamination)
Fishing	5 p.p.m. D.O., min.		Low D.O.; reduced fisheries resources, or fish kills

water uses would be considered in jeopardy. Since a reasonable margin of safety has been incorporated in the criteria, a "caution" area exists between the "satisfactory" and "danger" conditions. If concentrations are not controlled and pollution occurs, actual damage of the type listed in the right-hand column would result.

Graphically the criteria values and the concepts expressed in Table I are shown in Figure 3. The curve, rising from left to right, depicts increasing concentration of material which can pollute the receiving water. The horizontal line represents the criteria set by the control board. Every beneficial use of water imposes two values to each of its water quality criteria: (a) a threshold value at which the use is not significantly impaired by the concentration; and (b) a limiting value at which the beneficial use is, to all practical purposes, inhibited or destroyed. These two values are plotted on the graph. When concentrations are below the horizintal line the water is satisfactory for the indicated beneficial uses. Between the threshold and limiting values, quality of the water is questionable. This is the caution zone. At concentrations above the limiting value, actual damage can occur.

It should be recognized that the hypothetical case has been over-simplified. In an actual case, criteria in addition to those shown would very likely be given, especially to protect irrigation and fishing. Additional water uses probably would have to be considered. Seasonal variations in water use would dictate seasonal variation in water quality criteria. If during certain seasons the waters were not used for bathing, or if the fish could tolerate a lower dissolved oxygen concentration, the discharger could reduce his waste treatment costs by making greater use of the waste assimilative capacity of the river. There are many other variables which must be considered in actual cases to keep waste disposal costs at a minimum while, at the same time, affording adequate protection to the receiving waters.

The entire process revolves around the important principle of coordinating the interests and activities of others. Each step of the over-all program is designed to take into consideration and to protect the interests of all public agencies and private or-

ganizations concerned with the related problems of waste disposal and water use.

The California Water Plan

One phase of the process just described may have to be modified in light of the proposed "California Water Plan." In discussing the second step of the process, it was pointed out that, although the control boards consult with authorities in the field of water quality, the discretionary power and responsibility for establishing water quality criteria rests with the water pollution control boards. There will be a major exception to the last statement if the California Water Plan, when enacted by the Legislature, specifies water quality criteria for exported or other waters. To understand the reason for this exception, a brief explanation of the plan is necessary.

California has a critical water problem resulting from the fact that the northern one-third of the State is an area of surplus waters whereas the central and southern portions of the State are essentially areas of water deficiency. Three-fourths of California's water crop falls north of a line drawn through the City of Sacramento, yet three-fourths of the need for water is south of this line. It is simply a matter of poor distribution on the part of nature.

The solution to the ever-growing problem of water shortages in the southern two-thirds of the State is being developed in a water plan that proposes exporting water from areas of surplus to areas of deficiency. The ultimate exports and imports of water, as pictured in Figure 4 have been estimated by the Division of Water Resources in their studies of the proposed plan. The widely-publicized Feather River Project is the initial unit in the proposed California Water Plan.

The plan is being developed by the

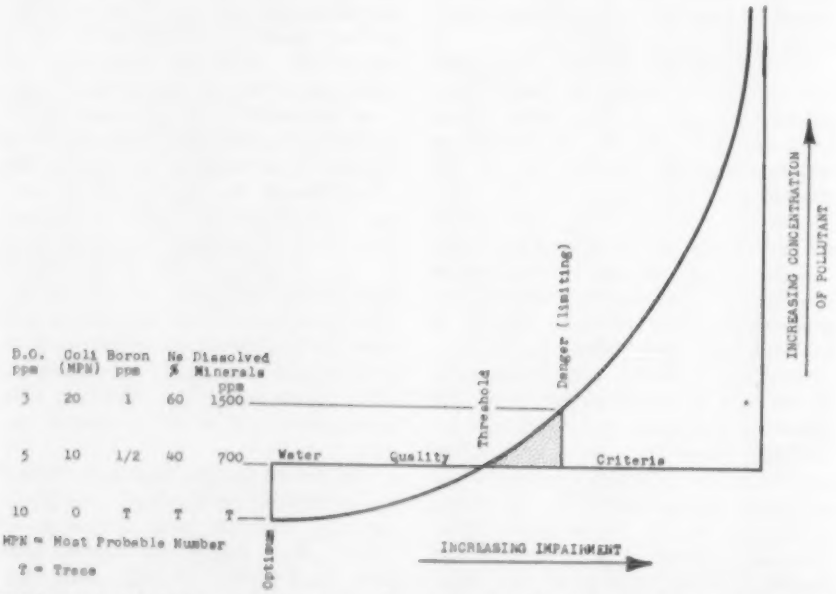

FIGURE 3.—Graphical representation of water quality criteria for each beneficial water use in a hypothetical case.

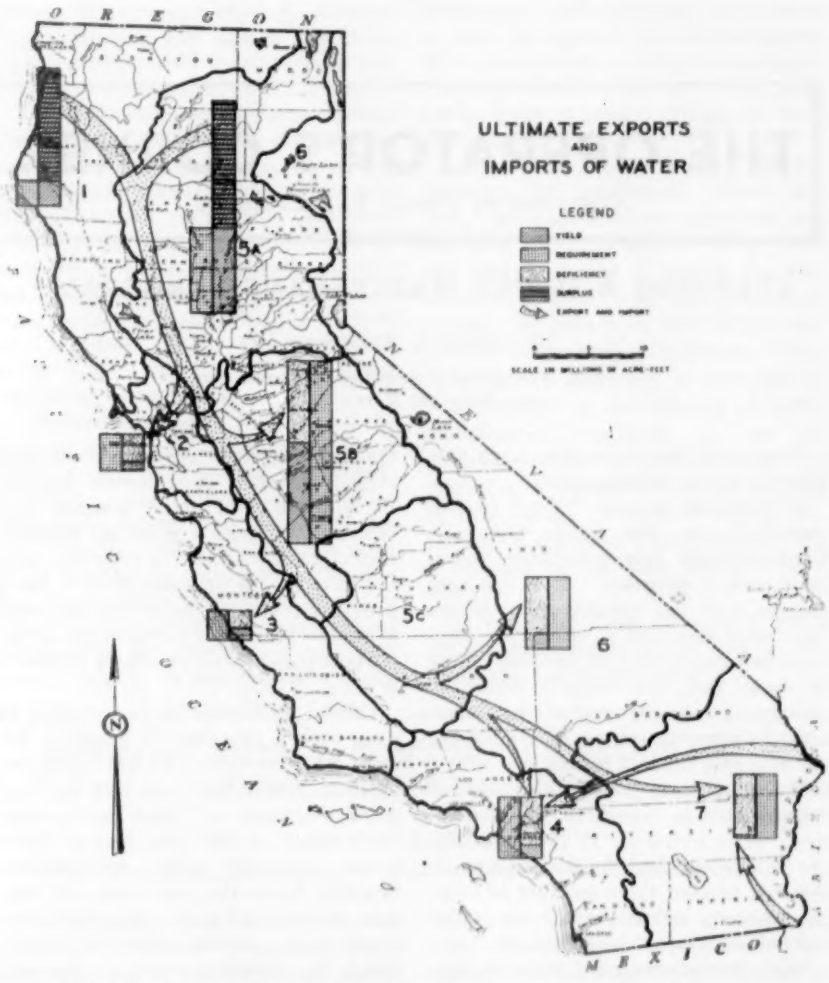

FIGURE 4.—The California Water Plan showing the hydrographic areas and water distribution.

State Water Resources Board pursuant to broad authorizations of the Legislature. This board, through the Division of Water Resources, is now engaged in conducting state-wide surveys and studies of water resources and ultimate water requirements. These studies encompass both water quantity and water quality. Upon completion of the proposed water plan, it will be recommended to the Legislature by the Water Resources Board. If the plan includes recommendations on water quality, and if the Legislature in adopting a California Water Plan fixes quality requirements for certain waters of the state, then these requirements become mandatory. The setting of water quality criteria on these waters would no longer be a discretionary responsibility of the water pollution control boards.

THE OPERATOR'S CORNER

CONDUCTED BY DONALD P. SCHIESSWOHL

STARTING A SEWER MAINTENANCE PROGRAM *

BY ORVILLE L. DICKINSON †

Chairman, Maintenance Committee, California Sewage and Industrial Wastes Assn.

Does your city or county have continuous sewer maintenance?

If you can answer "Yes" to this question you are most fortunate. Comparatively few governing bodies have such a program. Most are content to wait for trouble calls, before they send men out to clear the stoppage or other source of trouble. Most of these men are usually street or equipment operators and are not interested in sewer maintenance. They are not as a rule trained in this specialized field; as a consequence this type of maintenance is expensive and ineffective. It is irritating to the men who are called out and to the supervisor that has to send them out, not to mention the city officials upon whom the irate citizens vent their feelings.

Many jurisdictions fail to have continuous, preventive sewer maintenance simply because their personnel do not know how to organize such a program. Cities and counties pride themselves on the service rendered their taxpayers. Yet, key personnel fail to sell the city officials on the economics and value of this essential public service.

The record shows that the men who are successful in starting sewer maintenance programs advance themselves rapidly in their civil service careers. In addition to a better position and salary, they gain a personal satisfaction of achievement. If your city does not have such a program, or if it has a program that is dying on its feet, here is an oportunity for performing an important public service. Here is how it is done.

Official authorization is necessary to start such a program, so a selling job must be done first. To sell, facts are needed. Check back and find out how many stoppages or other emergencies have arisen in the past two or three years, especially night emergencies. Roughly figure the man-hours of time, both regular and night calls, that were required to correct these situations. Check the condition of the sewers each time a call is made. Is the main line flowing properly? Is the manhole structure sound, or is there deterioration? Check the pipe for its condition. With all these facts at hand, bring them to the attention of your superiors or other appropriate officials. Advise them that a program of continuous maintenance will eliminate trouble calls, restore the sewer to full capacity and prolong the life of the system. Enthusiasm on the part of the individual is an important part of the selling job. Help will be needed in

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† District Engineer, Clay Pipe Institute, Los Angeles, Calif.

getting the program started, because only a few men are able to stir their superiors sufficiently to get them to ask the city manager or council for funds.

The California Sewage and Industrial Wastes Association, through its local sewer maintenance sections and members of industry, has aided municipalities and responsible operating personnel in starting or revitalizing their sewer maintenance work. In the Los Angeles Sewer Maintenance Section, contact is made with the various city engineers. They are introduced to the Section and the activities of the California Association.

Efforts are made to hold one of the regular section meetings in different cities within the Section's territory. In this way, the various city engineers and other supervising personnel become familiar with the value of a good sewer maintenance program. Usually, the city engineer or director of public works is willing to start a maintenance program or improve the present program when he is acquainted with its advantages and economics, and when he sees the interest of his personnel. With genuine interest and cooperation from top official on down, the city governing body is usually willing to back the program.

Last but not least, look to private industry for assistance. There are many companies whose personnel are available for these efforts. They have the time to devote to telling the city officials the benefits of such programs. During the past year the author personally has appeared before no fewer than 12 city engineers or directors of public works in the interest of sewer maintenance programs. In the Los Angeles section alone there are four members of industry who are helping in this worthy project of selling continuous maintenance programs to the governing bodies who do not have them.

If your city does not have a sewer maintenance division, it is time to act. Start the ball rolling, now!

BENEFITS FROM A CONTINUOUS PROGRAM OF SEWER MAINTENANCE *

By N. F. Bravender

Sewer Maintenance Superintendent of Los Angeles County, Los Angeles, Calif.

Unfortunately most citizens are not aware of the great investment made through the expenditure of public funds for underground structures. Sewers and storm drains are usually covered by street pavement and being thus out of sight they are also out of the public mind, until something happens. Then the people ask "What is being done about the maintenance of these expensive installations?"

A continuing sewer maintenance program provides for the protection and proper operation of a sewer system. This program may be further defined as a program of continuous inspection of the sewer system, including appurtenances, covering each section with reasonable frequency to accomplish the early detection and prevention of sewer stoppages, deterioration, or faulty operation. It is distinguished from the indifferent programs of maintenance consisting of stoppage removal or repairs only when the system ceases to operate.

Programs of continuing maintenance exist in the more progressive and enlightened communities. There is a need for such a program in any com-

* Presented at Los Angeles Sewer Maintenance Section, California Sewage and Industrial Wastes Assn., Los Angeles, Calif. (1955).

munity that can boast of a sanitary sewer system. However, these programs do not come into existence by wishful thinking but by popular demand of alert citizens and competent governing bodies.

Policy must first be established. The governing body under the authority of existing laws determines the policy that shall regulate the program to be followed. Budgets, appropriations and available funds often limit the maintenance program whenever the governing body hesitates to make a tax levy sufficient to provide the necessary funds. In addition to salaries and wages, items of capital outlay for equipment must be provided. It has been demonstrated in this mechanical age that adequate equipment is the economical approach to greater production with the same labor cost.

The best intentions often fail to produce results because of the lack of leadership and competent supervision inspired with the responsibilities of public service. Someone must interpret the policy of the governing body, issue the orders initiating action and evaluate the results produced.

Maintenance Functions

The procedures and processes of maintaining a sanitary sewer system consist of the following functions:

1. The periodic and routine inspection of all sewer lines, manholes, pumping stations, or other appurtenances for the purpose of detecting the presence of any substance or condition that might result in a stoppage, deterioration of the structures, or the creation of a dangerous condition.

2. The cleaning of any sewer line, manhole, pumping station, or other appurtenance found, by inspection or otherwise, to be in need of cleaning.

3. The repair of any sewer line, manhole, pumping station, or other appurtenance found to be in a condition requiring repair or replacement of parts.

4. The testing of all structures for the presence of combustible gas, bad odors, toxic fumes, oxygen deficiency, or industrial wastes that might create hazardous conditions for the public or crews engaged in work below the street surface.

5. Cooperation with industry regarding waste treatment processes and the discharge of industrial waste which might be deleterious to the structures, dangerous to the maintenance crews or cause improper or excessive use of the pipe capacity.

6. The control of rats, cockroaches, spiders, or other vermin which infest the sewer.

Objectives

Many citizens say that it sounds like a good idea to have a sewer maintenance program but it is too extensive and would cost too much. Let us look into the matter to determine the responsibility of the governing body and the results that could be obtained for the money expended.

A maintenance program should have an objective. The results to be attained are the basis for the various operations and procedures. The justification of a program lies in the evaluation of results as evidence of funds well spent. Sewers and sanitation are usually classified under public health and safety, and it may be safely assumed that a sewer maintenance program is designed to protect the public health and safety. Certainly preventive maintenance will reduce the frequency of stoppages and the resulting overflow of sewage onto the ground surface. The odor and sight of sewage in the street is repugnant to the citizen who resides there or must cross the street. Children are prone to play in any stream of water and the parents become properly indignant when they learn it is raw sewage. Compaints to the health officer of filth, illness and disease soon reach other civic officials and the newspapers.

To prevent the contamination of the streets, the embryo stoppages should be found before they become complete stoppages. Root growths in the pipes must be found and removed in the early stages of growth before the removal operation becomes difficult and expensive. Equipment should be available on short notice to repair or replace broken pipes. Ordinances should be adopted requiring connection to the sanitary sewer when available.

Legal

Legal precedent has been well established under public liability acts whereby a city cannot escape its obligation to inspect the sewer system for defective operation. Failure to receive notification that a defective condition exists is insufficient excuse for neglecting to maintain the sewer in a proper operating condition. The fact that a sewer has been built places the responsible officials on notice that they should be diligent in making inspections for the discovery of defective conditions. Failure to observe this responsibility has cost many cities hundreds of dollars in judgments from damage suits, as well as severe criticism of the responsible city officials.

Cost

Sewer maintenance is worth the cost. In the fiscal year 1953–54 the sewer maintenance districts of Los Angeles County had an assessed valuation of approximately one billion dollars. The total annual cost of sewer maintenance in the districts was $250,550. For each $1,000 assessed valuation the cost of maintaining the sewer system was $.26 per year; a bargain in public service.

Public Attitude

The American citizen takes great pride in his home with all its modern comforts. Nothing can be more disgusting or even terrifying to the fastidious housewife than to have sewage overflow from the plumbing fixtures. Floors, rugs and walls can be quickly ruined, resulting in considerable loss to the owner. Here again adequate sewer maintenance is the proper preventive measure. Manhole covers should be accessible so they can act as relief valves, permitting sewage to overflow to the street before it overflows in the home. Backwater valves should be required by the plumbing ordinance where plumbing outlets are below street level. The entrance of rats and other vermin into the home through the sewer system brings immediate public reaction. Certainly the health of the public must be protected by proper eradication of such vermin.

The stench of a foul smelling sewer has caused many complaints, yet it has not always been the fault of the sewer condition. Many industries will contribute vile smelling wastes if not properly controlled. Clean sewers with proper ventilation will eliminate most odors and complaints.

Occasionally a sewer blows up inflicting personal injuries and property damage. Leaking gas mains or the discharge of petroleum products into the sewer create a combustible atmosphere which only needs a spark to set off a destructive explosion. Routine checking of the atmosphere in sewer manholes with an explosion meter will give early warning of the danger. The utility company will welcome information on leaks and gladly repair their pipe.

Traffic hazards created by loose manhole covers that tilt or have popped off the frame have caused many vehicular accidents. In addition to being dangerous, these covers are also noisy and disturb the nearby residents. They should be satisfactorily adjusted or replaced. Another traffic hazard is the cave in where the street surface collapses under the weight of a vehicle. Broken pipes often create partial stoppages which wash out the dirt above the pipe forming a large cavity under the pavement. Routine inspec-

tion and cleaning should disclose the broken pipe.

Protecting the Investment

What is done to protect the public investment in the sewer structures? Are uses prevented which could result in coating the interior of the pipe with a hard encrustation that reduces the capacity of the line? Are greases, tars, acids and solid matter eliminated from waste waters discharged into the sewer? Is industry encouraged to conserve water and reduce the quantity of sewage by recirculating the cooling water? Vigilence in these matters is a protection to the property and rights of the citizens. Toxic and poisonous material can upset the treatment plant processes and sludge digestion, resulting in higher costs and plant expansion. When it rains do citizens remove manhole covers to drain flooded streets, thereby introducing dirt, sticks and trash into the sewer or flooding out a pumping station? Do cesspool cleaning companies have unregulated use of the sanitary sewers? All these improper uses of the public sewers should be brought under proper control for the benefit of the general public and their investment.

Construction work by other utilities on many occasions results in damage to the sanitary sewer. Contractors installing storm drains, gas and water lines, telephone and power conduits, poles, tunnels and other sewers often damage an existing sewer. Good underground records showing the location of sewers will prevent much of this damage. Many sewer pipes are broken by street traffic when they are laid too close to the ground surface or by the crushing loads of high fills. Careless operations by various contractors, plumbers and grading contractors may result in damage to the sewer system. Damage by others involves considerable expense for repairs which should be collected from the parties responsible.

Normal use of a sewer system produces wear and deterioration of the facilities. The life of a sewer system can be extended considerably by the repair of broken pipes, broken manholes, removal of roots, grease and trash. Buried manhole covers should be raised to keep the manholes available for access to the sewer. Rusted manhole steps which have become dangerous should be replaced. The need for parts replacement in a pumping station should be kept in mind.

A good set of records and plans are essential to maintenance and the planning of future extensions or relief sewers. They also assist other utilities in steering clear of sewers when constructing underground structures. When lost manholes or wye branches must be located the records become invaluable.

Good public relations are desired by those engaged in public service. What kind of response or action does a complaining citizen receive? Is it indifferent or delayed, or does the citizen call or phone to compliment on the prompt and efficient service? A well equipped crew with a little resourcefulness can cope with most of the problems encountered. Does your program include in-service training and the passing on to the other crews the "know how" developed by one crew. Reading the trade journals is a good way to pick up new ideas. A crew that is familiar with the situation, has know how or makes a quick response to a complaint certainly has the endorsement and support of the citizens.

Frequently the citizen appeals for help on a personal basis. A ring is lost down the drain, the plumbing stops up and a plumber can't be located. Alarm and worry must be alleviated by sympathetic help in solving the problem. Remember, the taxpayer is really the boss. Keep him happy and satisfied within reasonable limits.

WASTE TREATMENT PLANT FEATURES

EDITOR'S NOTE: These articles represent a few of the interesting treatment plant features presented by a 10-member panel at the 1955 Annual Meeting, Michigan Sewage and Industrial Wastes Assn.; Charlevoix, Mich.; June 20–22, 1955.

PAPER MILL WASTES AS A "ROAD BINDER"

BY R. J. SUESS

President, Otsego Falls Paper Mills, Inc., Otsego, Mich.

Disposal of paper mill digester wastes as "road binder" has had several beneficial effects. Local street and highway departments have an inexpensive binder for alleviating dust problems or constructing secondary roads. An effective means for disposing the high B.O.D. wastes is readily available to the paper mill. Finally, the feasibility of this method of waste disposal has permitted the continued operation of a paper mill using local state hardwoods, thereby contributing to the economical welfare of the citizens.

Experiments several years ago revealed that a reasonably good quality paperboard could be manufactured from Michigan hardwoods which at that time were considered to be weed trees of no practical value as a forest product. While this venture was a sucess it did aggravate the problem of waste disposal. The major problem involved the discharge of high B.O.D. materials from the pulping process. These wastes were being discharged to the Kalamazoo River whose waters were already overloaded with waste materials before reaching Otsego.

FIGURE 1.—Loading "road binder" into tank trucks for application to streets and secondary roads.

FIGURE 2.—Tank truck applying "road binder."

For several years experiments had been conducted on a small scale to see if the strong digester waste could be used for road construction and maintenance purposes. This work proved so promising that the street department of the city of Otsego and the Allegan County Highway Department were asked to run some full-scale experiments on their streets and highways. These trials confirmed the original findings that this waste material would effectively settle dust on secondary roads for long periods of time. In addition, the waste would also bind the road materials together, thereby reducing maintenance.

The name "road binder" was adopted for this material and a full-scale program was immediately setup for its use. Evaporation or blow-down tanks were installed into which the material from the pulp digesters was pumped and partially evaporated. Large storage ponds were constructed to hold the concentrated waste during periods of bad weather or when the tank trucks could not apply it to the roads. The evaporation tanks and ponds were piped to a loading header (Figure 1) equipped to facilitate the loading of the tank trucks.

Several methods of applying the waste are used depending on the results desired. Many roads are given only a spray treatment for the alleviation of dust. (Figure 2). Best results have been obtained where the road has been scarified first, then graded and new materials added where needed prior to treatment with a sufficient quantity of road binder to properly coat the road materials.

The tank trucks hauling the road binder carry the wording "Using Otsego Falls Road Binder." In addition, the trucks publicize the stream pollution abatement features of the program by carrying the following phrase on the rear of the tank—"Doing Our Part to Clean Up the Kalamazoo River."

During the summer of 1954 the Alle-

gan County Highway Department and the street department of the city of Otsego used the road binder continuously in treating the highways and streets. No strong pulp digester liquor was discharged to the river, thereby accomplishing 100 per cent stream pollution abatement.

ECONOMICAL REFINERY WASTE DISPOSAL

By Charles E. Fiske

Technical Director, Leonard Refineries, Inc., Alma, Mich.

Leonard Refineries started operations in 1936 with a skimming plant processing about 3,000 bbl. of crude oil per day. The API separator constructed at that time to handle the effluent has gradually been overloaded as the refining facilities have expanded. In order to maintain a satisfactory oil content in the effluent, which was discharged to the Pine River, it was necessary to maintain a series of straw dams along a small stream through which the effluent water flowed prior to its entry into the river. Maintenance of these straw dams required about 60 man hours per week and in addition to being costly, they were very difficult to maintain during periods when the water level was high or the ground was soft along the banks of the stream. It, therefore, was decided that a cheaper and more reliable method should be found for the separation of oil from the refinery effluent.

Of the various commercial separators offered, the dissolved-air flotation process looked most attractive for three reasons. First, the installation could be a compact package unit, offering good clarification with only about 18 min. retention time at full flow. Second, although alum or other chemical feed was to be used, it was claimed that the dissolved-air flotation principle could yield a substantially better effluent water should the chemical system fail. Third, there was an excellent chance that the separated oily skimmings could be kept to volumes which would not constitute a further disposal problem.

Accordingly, a 500 g.p.m. dissolved air flotation unit (Figure 1) was purchased early in 1954 to clarify the refinery effluent. With the exception of one 8-hr. shutdown for the purpose of cleaning the retention tank and flotation chamber, this unit has been operating continuously 24 hr. per day, seven days per week.

Process Operation

The effluent from the API separators flows into a sump from which the main process pump takes suction. Air pulled into the system by an eductor, enters the system at the suction of the pump and is mixed with the water in the pump. The water then passes through a retention tank where alum is proportioned to the flow by a metering pump whose stroke is controlled by the air signal from the level controller on the inlet water sump. Back pressure on the retention tank is controlled by means of an air operated control valve arranged to keep the air dispersed in the water until the water reaches the flotation chamber. Retention time in the flotation chamber is from 18 to 35 minutes. Clear water is withdrawn from the chamber bottom and the floating sludge is skimmed off the surface of the water over an inclined weir at the end of the flotation chamber.

A rather thorough investigation was

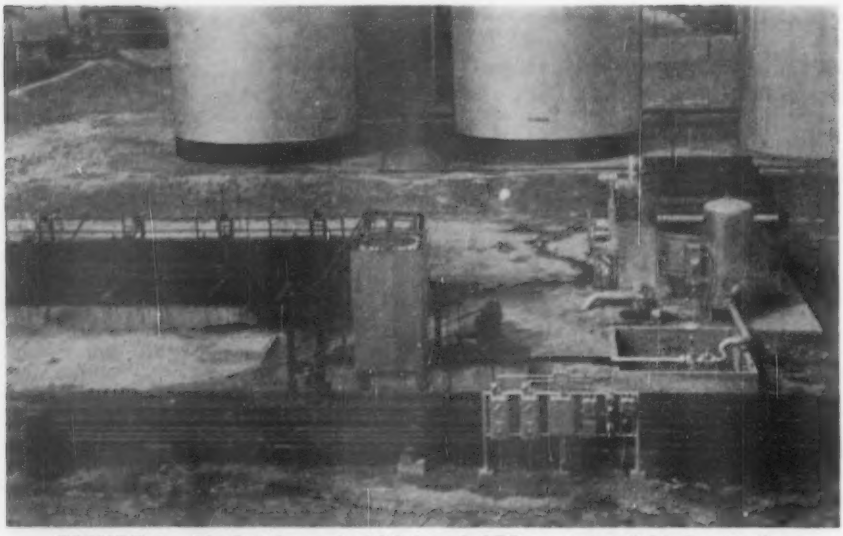

FIGURE 1.—Air flotation unit (right) and API separator (left) for treating refinery wastes.

made of the operating variables to learn the optimum operating conditions for this particular waste. The alum feed rate, flight scraper speed and flotation chamber level were varied, and the resultant effect on the effluent quality was observed.

The minimum alum dosage for satisfactory oil removal was determined to be 60 p.p.m. and the best results were obtained with lineal flight scraper speed of 16 in. per minute. When setting the flight scraper rates at the average flow of 181 g.p.m. the average retention time is 37.5 minutes. Since the skimming basin is 20-ft. long, this rate permits a minimum of turbulence at the skimmed layer while still being fast enough to prevent any settling of the floating sludge.

The volume of sludge removed from this unit has been relatively small and has not constituted any serious disposal problem. During the days of average flow approximately 1,000 gal. of sludge is obtained. This material is pumped to a pond area where the sludge is settled and the clear water pumped to waste.

Maintenance

Maintenance on this unit has been minor. Early difficulty was encountered with the rubber edges on the flight scrapers swelling out of shape, thereby causing the scrapers to do a poor job of skimming the sludge. This trouble was corrected by replacing the edges with a saran rubber formulation that is holding up well. Periodic routine maintenance is required on the pumps and instrumentation, otherwise the only trouble encountered was during a wet snowstorm. Snow froze and packed on the flight scraper rail, causing the flight scrapers to bend. This broke several of the brackets which held the scrapers and these had to be welded. Operation of the unit is completely automatic and the unit is unattended except for an hourly check by an operator who observes that the pumps and instruments are functioning properly. In case of pump failure an automatic signal attached to the float in the pump sump is sounded. A standby pump is provided.

A saving of about $10 per day has

been realized in the operation of the unit as compared to the cost of maintaining the straw dams. The quality of the refinery effluent water has been improved to the extent that serious consideration is being given to reusing this water as process water in the refinery.

KALAMAZOO OPEN HOUSE

By Donald H. Swets

Superintendent, Sewage Treatment Works, Kalamazoo, Mich.

Sometimes it takes a few years to progress from the laying of the first sanitary sewer until the dedication of the treatment facilities. Usually during the intervening years many economic and social changes take place which make a once accepted method of disposal entirely unsatisfactory.

This was the case of Kalamazoo, Mich., which laid its first line in 1881 and dedicated the treatment works almost 75 years later, in 1955. Great population growth, economic changes, world wars and finally a demand on the part of the citizens for adequate waste treatment marked the intervening years.

Treatment Facilities

Particular features of this modern sewage treatment installation (Figure 1) are the use of "coilfilters" for dewatering digested sludge and aeration-vacuation equipment for removing grease, oil and light solids. The treatment plant was designed to give a high degree of primary treatment to a flow of 12 m.g.d. from a population of 100,000. At present, the plant is receiving slightly more than 7 m.g.d. from the 60,000 persons served within the corporate limits. The sewage averages 200 p.p.m. suspended solids and 240 p.p.m. B.O.D.

Raw sewage arrives at the plant through a 54-in. and an 18-in. line which join at an overflow chamber. The sewage is comminuted and flows to the grit chambers and then into the wet well in the Main Building. Two variable speed gas engine driven pumps of 8 and 12 m.g.d. capacity and two electric driven units rated at 6 and 12 m.g.d. provide the necessary pumping capacity.

The first major treatment unit is the "vacuator." Air is mechanically entrained in the sewage which then flows into a domed structure where 9-in. of vacuum is applied to the surface for about 15 minutes at design flow. This causes the entrained air to rise bringing with it greases, oils and light solids which are then skimmed from the surface and pumped to the sludge digestion tanks. The unit has shown removals of about 20 per cent suspended solids and B.O.D. Volatile content of the solids is approximately 90 per cent.

Two 32 gal. per hour liquid feeders feed liquid ferric chloride into the sewage and two dry feeders supply the hydrated lime. Dual rapid mix units and the flocculation tanks provide a 30 min. flocculation period.

Two circular sedimentation basins with 85-ft. diameters provide two hours detention at design flow. The treated sewage is chlorinated prior to discharge to the Kalamazoo River.

Sludge and scum are pumped to two primary digestion tanks each of which has a capacity of 100,000 cu. ft. Optimum temperatures are maintained by the use of a pair of sludge heaters which can utilize the waste heat from the gas engines driving the pumps, or can be fired with oil or sludge gas. The two secondary tanks, equipped with floating covers, are comparable in capacity to the primaries.

The digested sludge is dewatered on a coil spring vacuum filter and the resultant cake is trucked to a storage

FIGURE 1.—The 12 m.g.d. Kalamazoo Sewage Treatment Works during the final stage of construction.

area. The city Parks Department and citizens of the community have shown an interest in this product, but as yet no arrangements have been made for final use.

Early operating results indicate a 65 to 70 per cent removal of suspended solids and a B.O.D. removal of 40 per cent. By adding chemicals to the sewage between the hours of 9 AM and 9 PM, using a dosage of 10 p.p.m. ferric chloride and 30 p.p.m. lime, the removals during several weeks of trial operation were increased to 80 per cent and 55 per cent for suspended solids and B.O.D., respectively.

Public Relations

An important feature of the program that culminated in the construction of the Kalamazoo sewage treatment plant was the excellent public relations carried on by the citzens and city officials most concerned with the waste treatment problem. Relations with members of the local press were exemplary before, during and after the construction of the plant. Newspaper support aided in obtaining public approval of the project an financing. Excellent coverage was given the actual plant construction.

As part of the over-all public relations program an attractive eight page brochure was prepared. Written for the average citizen, the brochure describes with clear wording and excellent photographs the equipment and features of the citizen's sewage treatment plant. With flow diagram in hand it leads him through the treatment plant from comminutor to chlorine contact chamber.

As a special feature an "Open House" was held July 9 and several hundred citizens braved rain and heat to get a look at the largest single project, dollarwise, that the city has undertaken. The brochure was distributed at that time and it is hoped the citizens of Kalamazoo will have a fuller appreciation of this vital public service.

Plans are already formulating to continue the already excellent public relations.

INTERESTING EXTRACTS FROM OPERATION REPORTS

"If you don't like life, it's the way you're living"—J. W. T. Meehan

Attention operators! It will be appreciated if copies of all annual reports of both large and small plants are sent to the Federation office for abstracting in this section of the Journal. Carbon copies of typewritten reports will be promptly returned on request. Please direct annual reports to: Federation of Sewage and Industrial Wastes Associations, 4435 Wisconsin Ave., N.W., Washington 16, D. C.

22nd and 23rd Annual Reports on the Aurora, Ill., Sanitary District Sewage Treatment Plant for the Years 1953 and 1954 *

By Walter A. Sperry, *Superintendent*

General

The weather during this two year period (1953 and 1954) showed considerable departure from normal. Low flow in the receiving river (Fox) and high summer temperatures in 1953 resulted in a serious fish kill. Investigation revealed that the pH and dissolved oxygen content were near normal and no industrial waste was involved, but water temperatures were as high as 108° F., literally cooking the fish. In complete reversal of this weather condition a record rainfall occurred in October, 1954, resulting in serious flooding of the treatment plant and extensive damage to interceptor sewers.

Raw sewage flow was less than during 1951 and 1952 when it averaged approximately 8.2 m.g.d. Flow for 1953 was 7.03 m.g.d., increasing to 7.75 m.g.d. the following year.

Both the B.O.D. and suspended solids of the raw sewage increased 24 per cent and 18 per cent, respectively, over the preceding year. This is due to concentration as a result of lower sewage flows. Overall removals were slightly better in 1953 and 1954, averaging more than 80 per cent in each case. Probably due to fewer days of secondary tank operation in 1954, the removal of suspended solids decreased slightly while the B.O.D. removal increased.

Digester gas production reached a high of 15.3 million cubic feet in 1953, an increase of 25 per cent over production in 1951 and 1952. The greater volume of gas is a result of an increase in primary and secondary sludge. The longer use of the filters and secondary clarifiers was partly responsible for more sludge. Gas production dropped 2.6 per cent in 1954, but remained relatively high due to a continuation of the 1953 conditions affecting sludge concentration.

The accident record for 1953 and 1954 was improved some. As a result of back injuries one employee retired after 9 years of service.

Improvements and Additions

During 1954 electric service was again supplied by the public service company after a period of 13 years. A steady 110 v. current will be available for laboratory instrument operation and auxiliary lights to replace kerosene lanterns. Cross-switching makes it possible to continue the use of plant-generated current.

Most important improvement due to electric service was the provision for using the existing 6 m.g.d. electric driven pump. This will permit pumping peak flows rather than discharging

* For last previous extract see This Journal, **26**, 4, 567 (April, 1954).

2 to 3 m.g.d. of primary effluent when only the 8 m.g.d. engine pump was used. Low river flows make complete secondary treatment desirable. Moreover, an engine installation would have cost about $8,000 whereas the electric power installation did not exceed $2,000. This included new switch gear and other electrical appurtenances.

As a corollary to operating the 6 m.g.d. pump, the air release valves on the wet well pumps were revised, making the priming of the pumps automatic and much quicker.

A minor but importat labor saving change was the substitution of an electric motor for a gasoline engine to operate the pump used when dewatering the clarifiers or filling the digesters.

The equipment company that furnished the preaeration equipment revised the aerators in 1954 by substituting special air nozzles for the dual air-water system and airlifts originally installed. Operation of the aerator was greatly simplified at no expense to the sanitary district, performance was improved considerably.

Major addition to the plant will be the construction of two fixed cover digesters similar in capacity to the existing digesters. This needed additional capacity has been planned for several years and contracts are expected to be let in 1955.

A hole worn in the clarifier house roof by the engine exhaust was repaired in 1953 and the exhaust shifted to discharge sideways rather than down. A 6 h.p. gas engine flap valve pump was added as a valuable piece of plant equipment, especially when cleaning digesters. The plant owned cottage was given a new roof, the first time in 20 years.

Maintenance

Usual yearly maintenance of plant and grounds included painting and repairing structures needing attention and the rolling of the lawn together with trimming shrubbery and planting perennials.

A severe tornado in May, 1955 required considerable extra work in clearing the grounds of fallen trees and other debris. Considerable repair to skylights and similar structures was necessary. In addition, the flood of 1954 required repairs to shorted circuits. Fortunately the prompt action of plant personnel and the height of equipment prevented serious damage to electric motors. The pump and electric switchboard room escaped flooding by one-half inch.

The trouble in the sludge pump (No. 2) due to heavy suction pulling the ball check through the rubber ring was satisfactorily corrected by substituting a bronze ring of slightly smaller diameter for the rubber ring.

No. 1 digester was cleaned in 1953 of a large accumulation of grit and sludge. In addition to this difficult task, the bottom plow hinges were found broken, necessitating removal and replacement of the bent drive shaft and the hinges.

The four gas engines are completely overhauled each year. The expensive and annoying maintenance job of retubing the exhaust heat exchanger was undertaken again in 1953. Wrought iron tubes are used and their life is approximately 3½ years. New exchangers operate in a more efficient manner, but plant conditions do not permit a change.

The 1,000 gal. plant water supply tank corroded through. This heavily insulated tank was impossible to move because of its size and location between floors. A tank lining company lined the tank with 1½ in. of waterproof cement in one day at a cost of $386. The work is guaranteed for five years.

Collection System

During the two years, 1953 and 1954, 2.16 sq. mi. of territory were annexed to the sanitary district for

a total of 15.6 sq. mi. Connections continued at a rate which reflected the active residential building. The construction of lateral and interceptors continued at a moderate pace. Major work cleared for construction in 1955 was 4.7 mi. of laterals.

The flood of 1954 required the replacing of considerable pipe in two interceptors at a cost of $16,000. Except for the flood, sewer and lift station operations were near normal.

Table I is a summary of the 1953 and 1954 operating data.

TABLE I.—Summary of 1953 and 1954 Operation of the Aurora (Ill.) Sanitary District Sewage Treatment Plant.

Item	Average 1953	1954
Rainfall (in.)	28.7	47.03
Tributary pop. (estimated)	60,000	60,000
Sewage treated:		
Total (m.g.d.)	7.03	7.75
Complete (%)	95.6	82.4
Screenings (cu. ft. per m.g.)	0.91	0.94
Grit (cu. ft. per m.g.)	0.89	0.64
Volatile (%)	64.1	61.8
Grease and scum (cu. ft. per m.g.)	89.2	90.0
Suspended solids (p.p.m.):		
Raw	206	198
Settled	84	86
Filter effluent	51	58
Final	28	34
Reduction (%)	86.6	82.7
B.O.D. (p.p.m.):		
Raw	81	86
Settled	55	56
Filter effluent	23	24
Final	16	15
Reduction (%)	86.6	82.7
Nitrogen, as NH_3 (p.p.m.):		
Raw	8.4	9.5
Settled	9.2	9.8
Filter effluent	3.3	3.8
Final	2.8	3.5
Nitrogen, as NO_3 final effl. (p.p.m.)	6.4	6.6
Raw sludge:		
Total solids (%)	6.1	5.9
Volatile solids (%)	64.8	63.2
Final settling tank sludge: *		
* To 2nd-stage digesters.		
Total solids (%)	5.2	6.3
Volatile solids (%)	55.2	49.3
Transfer sludge—1st to 2nd stage:		
Total solids (%)	2.85	2.83
Volatile solids (%)	50.5	46.65
Digested sludge:		
Total solids (%)	3.0	2.9
Volatile solids (%)	49.3	46.7
Supernatant (from 2nd-stage digester):		
Total solids (%)	1.7	1.1
Volatile solids (%)	48.9	47.8
Gas production:		
Total (mil. cu. ft.)	15.29	14.89
Cu. ft. per lb. volatile added	5.18	6.8
Cu. ft. per m.g. sewage	5,960	5,270
Cu. ft. per capita	0.68	0.70
Gas utilized (%)	92.5	86.6

TABLE I.—*Continued*

Item	1953	1954
	Average	
Gas composition:		
CO_2 (% volatile)	32.6	33.3
O_2 (% volatile)	0.4	0.4
H_2 (% volatile)	1.5	2.5
CH_4 (% volatile)	63.7	63.3
N_2	1.6	0.5
H_2S (p.p.m.)	129	142
B.t.u. (net)	580	578
Gas engine net earnings in year ($)	2,235.90	3,064.22
Plant operating costs ($):		
Payroll	51,929.00	54,534.19
Retirement fund	5,236.04	5,370.30
Interceptor system	3,681.13	6,021.91
Truck	203.17	244.72
Office	623.86	818.22
Laboratory	390.81	353.30
New equipment	1,137.91	2,005.05
Repairs	3,639.39	3,529.31
Gas engines	4,995.35	1,695.17
Electricity and gas	—	1,889.32
Miscellaneous	3,937.41	3,167.91
Total	74,873.67	79,629.50
Total cost per m.g.	29.19	28.16
Total cost per capita	1.25	1.33
Total disbursements, operation, administration, interest, bonds, etc. ($):		
Per m.g.	47.79	39.47
Per capita	2.04	1.86

INSTALLING SEWER SADDLES *

By John A. McCollum

District Foreman, Sewer Maintenance Division, Los Angeles, Calif.

A sewer saddle is that essential connection made between the house sewer and the sanitary sewer lateral or interceptor. All too frequently, however, this connection is the weakest link in the chain that provides effective service to the consumer. Installed by novices who lack experience or sound supervision, an improperly constructed sewer saddle can be a constant source of irritation to the sewer maintenance crew. Eventually it must be replaced.

Changing building activity is responsible for the necessity of installing saddles. Sewers may be designed originally for residential areas, which because of increased population are rezoned for business, as a result more laterals are needed to accommodate the new buildings. In the beginning the lots may have been large and later were subdivided, resulting in more connections to the existing lines. Saddles are used in preference to wyes because when wye branches are repaired it is advisable to replace three lengths of pipe in order to be reasonably certain that the connection will be as sound as the balance of the line. Since this requires a large excavation it becomes an expensive project. Sewer maintenance records show that the less expensive saddle operation, properly exe-

* Presented at 1955 Annual Meeting, California Sewage and Industrial Wastes Assn.; Riverside, Calif.; April 27–30, 1955.

cuted, will be as durable and effective as a wye.

Saddle Construction

On small vitrified clay lines saddle installation is relatively simple. There are many methods which can be used successfully. One method that has been found to be satisfactory is to hold the saddle in the desired position and then to mark with a piece of chalk the outline where the hole should be cut. The saddle should be used as a pattern so the hole can be cut in the desired shape. A sharp instrument such as a prospector's pick is a suitable tool for making the opening. The pick is small and light weight, and can be used to good advantage in a small excavation. The final trimming can be done effectively with a small Stillson wrench. The hole should be cut the full diameter of the saddle to provide a smooth transition (Figure 1). After the hole has been cut, the saddle should be held firmly in place with a generous amount of cement mortar. An additive that hastens the hardening of the cement should be used so the saddle will not be pushed out of its proper place.

When the saddle is securely in place, mortar should be used to plaster the inside where contact is made with the main sewer. This transition must be smooth (Figure 2) so the flow of sewage is not obstructed because if any rough spots remain there is danger that debris will lodge and cause stoppage.

If the ground is stable recommendation is made that concrete reinforcing not be placed entirely around the existing pipe. It has been found that disturbing the original foundation and adding weight to the stop of the pipe will cause failure at nearby joints.

Saddles on concrete sewers up to 18-in. in diameter are installed in much the same manner as with vitrified clay pipe, except that it is usually

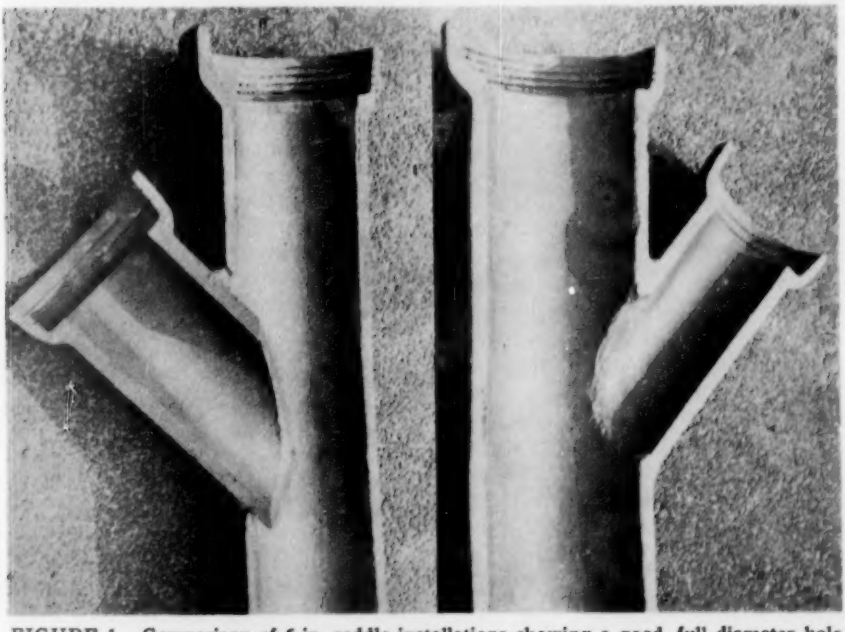

FIGURE 1.—Comparison of 6-in. saddle installations showing a good, full diameter hole (right) and an inadequate cut (left).

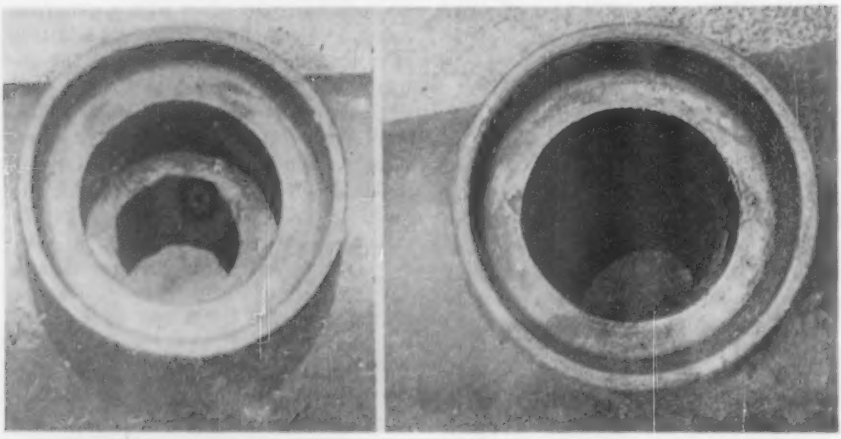

FIGURE 2.—Good and poor 6-in. "wye" saddle installations. Note smooth transition (right).

necessary to place a steel band around the sewer to reduce the danger of shattering when the hole is cut. The generous use of cement in making the joint is advisable because these lines are apt to be thinner shelled than other types of pipe. This additional cement should be placed along the existing pipe and used only sparingly on the top. Here, again, any increase in weight creates a dangerous situation.

When entering concrete sewers larger than 18-in. in diameter or when large pipes have tile liners, the hole should be made with a coring machine. If a pavement breaker is used the concrete is broken unevenly and tile is dislodged, allowing sewer gases to attack the exposed spots. Often failures develop, resulting in expensive repair jobs. In all cases teagle, tar or other gas resistant substances should be used to fill any voids left in the wall of the pipe. When a coring machine is not available a rock sinker drill may be used. A series of holes should be drilled and the core lifted out in one piece. This will keep most of the debris out of the main line, and having a clean pipe is one of the goals. The use of the proper equipment will reduce uneven breakage and loss of tile and lessen the chances of pipe failure.

The coring method is by far the best, and if it is not financially prudent to own such a machine one can be rented for a reasonable sum. Any money expended in this manner will be more than repaid by the extra years of life it will add to the pipe lines.

Responsibility for Installation

With the growing importance of preventive maintenance programs there should be no doubt that this must be done by the personnel whose job it is to keep the sewer system in the best possible operating condition. Sewers are a necessary utility and a valuable investment. Accordingly, they should not be experimented with by those without adequate training. When sewer taps are left to the average plumbing or construction contractor anything can happen, and sometimes does. Cases can be cited where the contractor placed a saddle on a telephone conduit. It was several weeks before this situation was discovered and by that time a definite health hazard had developed. It is imperative that the health of a community should not be jeopardized by such incidents. Immediately after this episode the particular sewer mainte-

nance division began to install the connections.

This is not the only reason that maintenance men should do all the tapping of the main lines. Before modern cleaning methods were developed a saddle protruding into the sewer line from one to three inches was rarely noticed. Pipes were seldom cleaned, and then usually by haphazard methods. When stoppages occurred they were opened, but the line was not really cleaned. Perhaps extra water was added with a fire hose so it would wash some of the debris downstream into a larger line where a stoppage was less likely to occur. With the present day corkscrews, balls, bucket machines and other cleaning equipment a pipe free from obstructions is needed to do a thorough job (Figure 3). When cleaning with a ball it is most discouraging to be unable to get the ball from one manhole to another and to discover, by means of a light flashed through the line, that a saddle is protruding several inches into the pipe. When bucket machines are used it is possible to exert enough pressure on the obstructions to break them, but often the main line is broken requiring expensive repairs. It is understood that many cities inaugurating a cleaning program had to practically discontinue it because of the many obstructions in the line.

Another source of trouble is the restriction of flow. When engineers design lines they know that a certain size pipe on a certain grade will carry so many second feet of water. Therefore, if inefficient workmen allow taps to protrude into the normal flow the velocity will be reduced and the amount of sewage passing through the lines will be less. As a result, 12-in. lines often have only the capacity of a 10-in. line, or an 8-in. line will have the capacity of a 6-in. line. With this

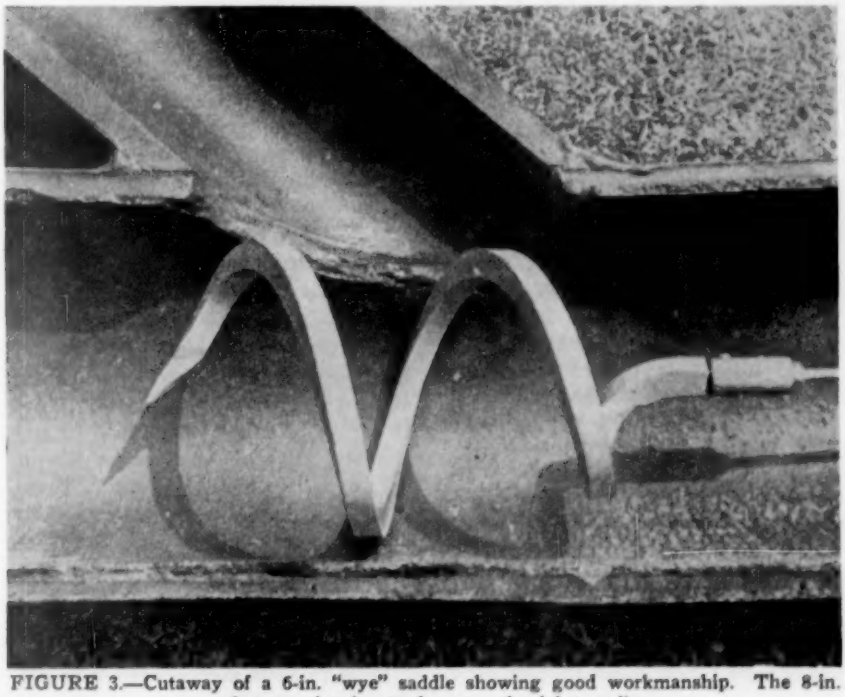

FIGURE 3.—Cutaway of a 6-in. "wye" saddle showing good workmanship. The 8-in. corkscrew cleaning tool passes the joint easily.

insufficient capacity new lines must be constructed to meet the needs, which adds extra expense to the taxpayer. Maintenance crews must be able to properly clean lines if the designed capacity is to be retained.

Recently, much experimenting has been done in the inspection of lines by actually photographing the interior of the pipes. A camera is mounted on a small boat attached to a light weight cable and the boat is then floated downstream. A continuous picture can be taken or the camera can be stopped at intervals for still shots. Protruding saddles might sabotage this project by upsetting the boat and causing the loss of valuable equipment.

The cost of any saddle should be paid for by the property owner because he alone will benefit from its use. It would not be fair to have the city, as a whole, bear this expense. The City of Los Angeles is now considering a charge based on the number of square inches of area of the entering pipe. At the present time a fee of $15 is required for lines up to 18-in. in diameter and $30 for those larger.

Conclusion

Every phase of sewer saddle installation has not been covered. On the contrary saddle installation techniques are a diversified subject widely discussed. These techniques are sometimes governed by changes in civil regulations and ordinances.

In the interest of over-all economy of maintenance operations and to insure cleaner lines and a reduced health hazard, sewer taps should be made by qualified personnel of the governing agency. The cost of this work should be paid by the individual benefited.

TIPS AND QUIPS

Rainbow Coatings

A unique method of corrosion protection has been reported by the Graver Water Conditioning Co., N. Y. Several excellent coatings in color have been developed by the paint industry for the protection of sewage treatment equipment. However, it is the reapplication of the paint that sometimes presents a problem and it is not always possible to determine how much protection has worn off or whether certain parts have suffered more wear than others.

To combat this difficulty the recommendation is made that several coats of paint (5 to 7) be applied at installation time, using successively different colors for each coat. As the protective coating wears off, the operator can tell at a glance, from the color showing, just how much wear has occurred, how much protection remains and which parts or sections have received more wear than others. He will know when recoating of all or part of the equipment is necessary.

Loyal Order of Manhole Sniffers

The dry cereal companies and other businesses have long competed for consumer attention through offers of one kind or another. Now we have the offer of a free certificate in the "Loyal Order of Manhole Sniffers" by the Bionetic Division of Reliance Chemical Corp., Box 6724, Houston 5, Texas. Bionetic is that persistent combination of preserved cultures of beneficial micro-organisms of uniform potency, claimed as first aid to waste treatment, and, incidentally, the subject of much pro and con discussion.

Send in your manhole cover or rea-

sonable facsimile today and be the first sniffer in your community.

Ice

Ice and snow will soon be tightening their wintry grip on sewage treatment plants in northern climes, if it hasn't already. An aid to keeping steps and walkways safe and slip free for visitor and employee is the use of a chemical ice melting compound.

An improved compound, Melt, is reputed to have 30 times the thawing capacity of salt. It quickly dissolves ice and snow without leaving a residue or harming vegetation, rubber, asphalt, or concrete. A phosphate-type rust inhibitor incorporated with the pellets protects metal from deterioration.

A special industrial grade snow melt granules X-73, is little affected by air moisture. It is ideal for mixing with abrasives for extra slip-proof protection. Both materials may be obtained from the Monroe Company, Inc., Cleveland, Ohio.

Public Relations

Calling attention to service as it is rendered, Village Manager, Mark E. Keane, of Oak Park, Ill., has placed first emphasis on sewer cleaning and forestry operations. When a sewer cleaning crew is scheduled to clean the sewer on a particular street, one crew member is assigned to place a brochure in each mail box of the residences fronting on the street.

The brochures are attractive, colored, one-page folded sheets, 8½ in. by 11 in., entirely prepared by village employees and runoff on the village multilith. Objective of the program, how the citizen can cooperate, progress of the program, whom to contact for service, source of money for the work and other items of interest are described in the brochure.

Sewer crews have also been supplied with tripod signs to be placed on the job site. The sign has the following caption:

"Getting the Most Out of Our Sewers"
Sewer Cleaning Crew
Village of Oak Park
Financed by Your Quarterly
Sewer Bill Payments

Gloves for Pipe Handling

Sewer maintenance and construction crews can obtain additional wear and safety in handling clay pipe by wearing appropriate gloves. The Edmont "Grappler" gloves, coated on the palm side only with a tough vinyl film, outlast heavy-duty canvas gloves five to one. The positive grip of the vinyl surface is a marked aid in handling the smooth pipe. Specialist in gloves of all types, the Edmont Manufacturing Co., Coshocton, Ohio, is able to handle inquiries for a single special glove or a carload.

Hurricane Precautions

The season of violent winds is over, but the damage is still in evidence throughout the areas that felt the full force of the hurricanes. Since these counter-clockwise revolving winds have taken an increased interest in a larger portion of the Eastern United States the following precautions should be kept in mind for use next season.

Briefly they are:

1. Stay off streets.
2. Don't touch wires.
3. Raise and securely tie or remove awnings.
4. Secure shutters as quickly as possible.
5. Brace against movement garage doors and other large doors.
6. Provide temporary, effective lighting methods.
7. Remove or raise to a safe level equipment which might be inundated or carried away by high water.
8. Open one or more windows on lee side to equalize air pressure.
9. Secure loose material and movable equipment.
10. Have a plan of operation and know what to do and when.

Reviews and Abstracts*

The Measurement of Sewage Flow Using Radioactive Tracers. By G. A. Truesdale. *Jour. Inst. Sew. Purif.* (Brit.), 97 (1953).

Experiments described show that the flow of sewage through settling tanks can be traced satisfactorily by adding Rb^{86}. By this method the long "tails" of the flow-through curves, which may be undetected by other means, can be determined. Wide variations were observed when salt was used.

Preliminary experiments with Rb^{86} in tracing flow through trickling filters are encouraging, but work using other isotopes and with compounds containing radioactive atoms is needed. H. P. O.

Treatment of Sewage by Alternating Double Filtration with and without Settlement of the Primary Effluent. By T. G. Tomlinson and H. Hall. *Jour. Inst. Sew. Purif.* (Brit.), 106 (1953).

Only a slight reduction in final effluent quality results when settlement of the primary effluent is omitted in alternating double filtration. The most striking effect of omitting intermediate sedimentation is to cause an increase in the amount of solid matter discharged from the secondary filter. The omission may be expected to cause some deterioration in the condition of the filters during the winter, although there was little evidence of this in the experimental plant. H. P. O.

The Experimental Treatment of Steamed Gas Liquor on Percolating Beds. By H. H. Goldthorpe and G. Taylor. *Jour. Inst. Sew. Purif.* (Brit.), 353 (1952).

Preliminary experiments showed the improvement in trickling filter treatment possible when crude gas liquor is pretreated by steaming before admission to the sewers. Gas liquors containing 10,000 to 15,000 p.p.m. total NH_3 and 9,000 to 15,000 p.p.m. permanganate oxygen demand were investigated. The crude gas liquor discharged as steamed gas liquor is at least 50 times as costly as primary effluent to treat on filters. H. P. O.

Some Experimental Work on the Effect of Synthetic Detergents on Sewage Treatment. By John Hurley. *Jour. Inst. Sew. Purif.* (Brit.), 306 (1952).

This paper reports large-scale experiments at the Coven Heath (Wolverhampton) sewage treatment plant concerning the effect on filtration of Teepol, a detergent of the secondary alkyl sulfate type. Also reported are experiments on the effect of Teepol on the activated sludge (Haworth) process at the Barnhurst plant.

With the filters at Coven Heath receiving their normal dosage of primary tank effluent, added Teepol (up to 206 p.p.m. in the raw sewage) caused no operational troubles and had no visible effect on the filters or on the appearance of the subsurface filtering medium. The receiving stream showed neither froth nor any other ill effect. Fish were frequently seen around the outfalls.

When the filters were dosed at double the normal rate, additions of up to 100 p.p.m. of Teepol temporarily increased the effluent B.O.D. by some 7 p.p.m. during the first month, but thereafter the treated and control effluents were consistently close in B.O.D. With this exception, dosing the sewage with 100 p.p.m. of Teepol had no important effect on any stage of the treatment.

In experiments to determine the progressive removal of Teepol by full-scale biological filters, the presence of up to 150 p.p.m. of the detergent with normal

* Please send to Federation headquarters all periodicals, bulletins, special reports, etc., which might be suitable for abstracting in This Journal. Publications of public health departments, stream pollution control agencies, research organizations, and educational institutions are particularly desired. Address such material: Federation of Sewage and Industrial Wastes Assns., 4435 Wisconsin Ave., N.W., Washington 16, D. C.

flow did not perceptibly retard purification at bed depths of 1, 2.7 and 4.3 ft., most of the Teepol being removed in the top foot of the filter. At double flow rates up to 100 p.p.m. of Teepol was satisfactorily removed, although the removal was accomplished deeper in the medium.

Addition of 125 p.p.m. Teepol to the settled influent to the Haworth aeration tanks at the Barnhurst plant (equivalent to 160 or 170 p.p.m. in the raw sewage) showed no noticeable effect except to increase the nitric nitrogen in the effulent to 5.5 p.p.m., as compared with 2.2 p.p.m. for the control without Teepol. Up to this dosage, however, the tendency was in the opposite direction. Even at the maximum Teepol dosing rate there was no noticeable difference in the amount of frothing in the dosed and undosed sections of the plant, other factors than the detergent seeming to exert greater influence on this phenomenon. H. P. O.

The Design and Construction of a Handling and Treatment System for Liquid Radioactive Wastes. By W. L. Wilson. *Proc. Inst. Civil Eng.*, 4, Part III, No. 1, 1 (Apr., 1955).

The paper describes the handling and treatment of radioactive effluent at Harwell, England. The principles for achieving segregation and economy in effluent discharges are developed; including the methods used for flow control, examination of discharges, and the basic function of the delay tank. Requirements for purity of discharge to the River Thames are outlined.

Experience with other wastes indicated that precipitation processes might be more desirable than distillation or ion exchange. The first problem was to reduce the volume of radioactive effluent to a minimum. Three systems of sewers were planned: the radioactive drain, the clean process drain, and the sewage and storm water disposal system.

Effluent requirements were:

1. Temperature not to exceed 60° F.
2. B.O.D. and solids in suspension not to exceed 20 p.p.m. and 30 p.p.m., respectively.
3. pH value within limits 6.5 and 7.5.

Approved concentrations of radioactive materials for drinking water were:

1. Alpha emitters
   Radium—4×10^{-10} micro curie per cc.
   Other alpha emitters—2.4×10^{-9} micro curie per cc.
2. Beta emitters
   Radiocalcium and radiostrontium—2×10^{-8} micro curie per cc.
   Other beta emitters—10^{-6} micro curie per cc.

The River Thames is the source of water supply for 8 to 9 million people. It has a minimum dry weather flow of about 50 m.g.d. (Imp.) Storage tank units were sized for an output of 300,000 g.p.d. (Imp.) and were built to be acid-resisting. Control of pH was regarded as a governing factor.

Precipitates were at first thickened and filtered. Later technique provided protective containers. Clean-process waste was available for dilution of the treated radiochemical effluent. Langdon Pearse

Operational Experiences with a Handling and Treatment System for Liquid Radioactive Wastes. By R. H. Burns. *Proc. Inst. Civil Eng.*, 4, Part III, No. 1, 21 (Apr., 1955).

Operational experiences with the Harwell, England radioactive effluent are summarized. Necessary alterations and modifications in procedure are detailed.

The maximum permissible levels of activity of the effluent, the source of this activity, and the various drainage systems are described. Necessary rigid controls and methods of treatment are discussed. Some of the operational difficulties and their solution are detailed.

Chemical dosing, followed by clarifloculators, has proved satisfactory. The thin slurry resulting is dewatered by a Sweetland press or a Bird centrifuge.

The discussion by various speakers was informative. W. L. Wilson defended the use of the clarifloculator and stated the capital costs of the works at Harwell was high. The effluent had a low B.O.D. and suspended matter content. The limits for discharge were far below those considered safe for workers in the atomic energy field. Langdon Pearse

Benefits to the Filters by Slower Dosing. By H. Hawkes. *Surveyor*, 114, 147 (1955).

The data presented in this paper were collected in the experiments described by

Tomlinson and Hall (*Surveyor*, **114**, 145). The film accumulation was much heavier with more rapid distributor rotation, although the vertical distribution of film was uniform through the top 2 ft. of medium in all cases. The larvae of *Anisopus fenestralis* and *Psychoda* species were correspondingly more numerous in the heavier film. *Lumbricillus lineatus*, on the other hand, was more numerous with the lower rates of distributor rotation. RAND

Industrial Waste Treatment at a Viscose-Rayon Factory. BY J. W. HUGHES. *Jour. Inst. Sew. Purif.*, 406 (1951).

Waste products from a typical large modern viscose-rayon factory are divided into solid, gaseous and liquid classifications. The first class is leached if obnoxious and dumped if not; the second, is recovered if market demand is sufficient, or treated to minimize air pollution if not, the ultimate object being sulfur recovery; the third group is accorded more detailed attention.

In the particular factory described, more than 3 m.g.d. of water is used and wasted. Liquors are utilized to their fullest extent wherever possible, consistent with economic considerations. Thus the final discharges consist of various liquors unsuitable for recovery by reason either of extreme dilution and/or contamination.

The drains segregate the wastes into four types: storm, sanitary (kitchens, lavatories, etc.), acid, and sulfide. Separate treatment plants handle the flows from the last two. "Acid" wastes actually are both acid and alkaline and may contain small quantities of metals, dyestuffs and oils. Treatment consists of neutralization, flocculation, settling, and sludge drying and dumping. "Sulfide" effluent is treated by bio-filtration, using one part of raw liquor to two parts recirculated liquor. The combined treated effluents are then given breakpoint chlorination to inhibit fungus growths in the long plant outfall to the public sewer.
H. P. O.

Biological Treatment of Cyanides, With and Without Sewage. BY A. E. H. PETTET AND E. V. MILLS. *Jour. of Appl. Chem.* (Brit.), **4**, 434 (1951).

The purpose of the experiments was to ascertain the concentration of metal cyanides that can be tolerated in the treatment of sewage by trickling filters. The cyanide complexes of zinc, cadmium, copper, nickel, and iron were used in this study in addition to potassium cyanide. The filters 4-ft. deep with 6-in. diameters were filled with ½- to 1-in. coke.

The results showed the first addition of 1 to 2 p.p.m. cyanides applied to a matured bed interfered with nitrification and increased the B.O.D. of the effluent, but the filter regained its efficiency in about a week. No cyanides could be detected in effluents from acclimatized filters treating sewage containing potassium cyanide up to the equivalent of 4 p.p.m. HCN or cyanides of zinc or cadmium up to the equivalent of 1 p.p.m. HCN. The experimental filters could be acclimatized to concentrations of at least 100 p.p.m. of HCN. At this concentration 99 per cent or more of the cyanide in potassium cyanide or in the complexes of zinc and cadmium could be destroyed whether dissolved in sewage or tap water. Similarly, 75 per cent or more of the cyanide in cuprocyanide or nickelcyanide could be destroyed. Purification of sewage was diminished but not inhibited by 100 p.p.m. of any of the cyanides used.

When the addition of cyanide to sewage treated in an acclimatized filter was stopped, the filter retained its resistance to the effect of cyanide for two weeks and its capacity to destroy cyanide for five weeks. A flora capable of destroying potassium cyanide can be developed under natural conditions without sewage. Biological pretreatment of cyanide waste before discharge into sewers is thus possible and offers an alternative to chemical oxidation. The highest rate of treatment tried in the absence of sewage was 0.1 lb. HCN per cu. yd. of filter media per day. H. HEUKELEKIAN

Sewage Works Design: Some Important Considerations. BY JOHN HURLEY. *Jour. Inst. Sew. Purif.* (Brit.), 418 (1951).

Among the factors discussed broadly as being important in sewage works design the author deals with effluent standards, local conditions, consistency of performance, treatment of storm water, ease of maintenance, degree of supervision, large-scale experiments, trade effluents, operator convenience, over-all coherence, provision for servicing and repair shutdowns, solids

removal efficiency, and recirculation effects. Discussion brought out the factors of sludge storage and disposal, flow velocities in various units, and conduit ventilation. H. P. O.

Some Historical Aspects of Sewage Purification in Scotland. By JOHN A. WARREN. *Jour. Inst. Sew. Purif.* (Brit.), 431 (1951).

This historical account of sewage treatment in Scotland begins with an extension of the sewers of Edinburgh in 1760 to Craigentinny Meadows on the shores of the Forth Estuary and disposal of the sewage by broad irrigation of hay and rye. The various developments are discussed briefly from then to the present time. H. P. O.

The Condition of the Tidal Portion of the River Thames. By C. J. REGAN. *Jour. Inst. Sew. Purif.* (Brit.), 85 (1952).

This is the original of a previously published condensation (see abs. THIS JOURNAL, 25, 1, 117 (Jan., 1953).

Studies on Acid Precipitation at Salford. By T. STONES. *Jour. Inst. Sew. Purif.*, 361 (1952).

Acid precipitation of sewage containing industrial waste is a function of the pH value and is independent of the nature of the acid employed. Optimum precipitation occurred at pH 3.0, which requires subsequent neutralization prior to biological treatment. H. P. O.

Recent Studies on Toxicity and Stream Pollution. By I. R. H. ALLAN, J. S. ALABASTER, AND D. W. M. HERBERT. *Water & San. Eng.*, 5, 3, 109 (1954).

The 10 m.g.d. of effluent from the Colne Valley activated sludge sewage treatment plant are discharged into a channel 0.875 mi. long before joining the River Colne. In the channel it receives little or no dilution. This channel contained large numbers of fish, and the D.O. goes down to 1 to 2 p.p.m. and ammonia concentrations of 15 to 30 p.p.m. are found. The reactions and distribution of fish in the effluent channel were studied (1) by confining fish of different species in aquaria on the bank through which effluent from a fixed point was made to flow, (2) by confining them in floating lineboxes in the channel and (3) by trapping and marking fish in different parts of the channel at different times of the year. The first two methods gave similar results, so that the second method was used because of its convenience. Immediately below the point of discharge of the effluent fish (roach) survived for long periods. The D.O. at this point was high. At the lower end of the channel where the D.O. was much lower captive fish died after a short period of exposure. The deficiency of oxygen, however, was not the cause of death of fish on some occasions at this sampling point. The results of the study dealing with the free swimming fish population showed that the effluent supported a large and varied coarse fish population (perch, roach, and gudgeon). Dead fish were found when lethal conditions occurred in the channel only occasionally. The number of fish in the effluent channel was small in the winter when D.O. was high and large during the spring and summer when D.O. was low. The absence of fish during the winter could not be attributed to adverse chemical conditions, but rather to migration.

The effluent channel contained a rich supply of fish food such as *Chironomid* and *Asellus*. H. HEUKELEKIAN

Agitation-Aeration in the Laboratory and in Industry. By R. K. FINN. *Bacteriological Reviews*, 18, 4, 254 (1954).

A review with 113 references devoted to the basic phenomenon of aeration as applied to fermentation industry. H. HEUKELEKIAN

Studies on Methane Bacteria—In Fixation of Atmospheric Nitrogen by Methanobacterium Omelians. By MARTIN J. PINE AND H. A. BARKER. *Jour. Bact.*, 68, 5, 589 (1954).

The authors were able to show that *Methanobacterium omelianskii* fixes nitrogen. Fixation was enhanced by ephemeral levels of NH_3 and abolished by more persistent concentrations. H. HEUKELEKIAN

Policy Decisions on Sewage Works Design. Institute of Sewage Purification. By H. H. STANBRIDGE. *Surveyor*, 114, 449 (Apr. 23, 1955).

Discussion of the influence of legislation on prevention of river pollution from the British standpoint of seven factors: (1)

the degree of centralization of sewage purification; (2) the location and layout of the sewage works; (3) the selection of the processes to be used for sewage treatment, including modifications of existing processes or the use of supplementary processes; (4) the removal of bacteria; (5) the treatment of storm water; (6) the amount of sludge to be dealt with; and (7) the location and design of effluent outlets. Nine references are appended.

LANGDON PEARSE

The Practical Gauging of Dirty Water and Its Applications to Sewer Design. BY S. F. MORT. *Proc. Inst. Civil Eng.*, 4, Part III, No. 1, 83 (Apr. 1955).

The paper describes the technique used during a period of 25 yr. by the Metropolitan Water, Sewerage, and Drainage Board of Sydney, Australia, and consists of (a) gaugings in sewers and open channels; (b) uses to which these gaugings may be put in hydraulic design, including sewage and industrial wastes. The author concedes that sewage and clean water have very similar hydraulic properties.

For primary effluent, the Parshall flume is preferred, and for secondary effluent, an integrating recorder. Diagrams and photos are shown. The applications of gaugings are described. Since 1935 the Board has obtained over 35,000 daily charts at more than 800 stations and adds about 20 charts per day.

The peak dry weather flow rate is determined. In addition infiltration and equivalent population (flow) are measured. The average domestic sewage ranges per day from 30 gal. (Imp.) in the working-class areas to 60 gal. (Imp.) in the wealthy districts.

The paper contains considerable tabular information. LANGDON PEARSE

BOOKS, REPORTS, AND PAMPHLETS

Proceedings of Third Southern Municipal and Industrial Waste Conference. The Technical Press, Raleigh, N. C. 322 pp. Price, $1.50. (1955).

This contains the transcript of the papers and discussions presented at the meeting, held at North Carolina State College on March 18–19, 1954.

Standard Specifications for Fluosilicic Acid. AWWA B703-54T. Amer. Water Works Assn., New York, N. Y. (1954).

Surface Water Resources of Georgia during the Drought of 1954. Part 1. Streamflow. BY M. T. THOMSON AND R. F. CARTER. Information Circular No. 17, State Div. of Conservation, Atlanta, Ga. (1955).

Proud Heritage. Div. of San. Eng., Mass. Dept. of Publ. Health, Boston, Mass. (1954).

An interesting historical document describing the activities and achievements of the Lawrence Experiment Station since its inception in 1886.

National Plumbing Code. ASA A40.8-1955. Amer. Soc. Mech. Eng., 29 West 39th St., New York 18, N. Y. (1955).

85% Magnesia Insulation Manual. Magnesia Insulation Mfrs. Assn., 1317 F St., N. W., Washington 4, D. C. (1955).

Steel Structures Painting Manual, Vol. II. Systems and Specifications. Steel Structures Painting Council, 4400 Fifth Ave., Pittsburgh 13, Pa. (1955).

Major Activities in the Atomic Energy Programs—July–December 1954. U. S. Atomic Energy Comm., Washington, D. C. (Jan. 1955).

The Defense Materials System in Our American Industry. U. S. Dept. of Commerce. 41 pp. Price, $0.25, from Supt. of Doc., Govt. Printing Office, Washington 25, D. C. (Feb. 1955).

Soda Ash Slurry Storage Systems. Diamond Alkali Co., 300 Union Commerce Bldg., Cleveland 14, Ohio.

Hydraulics Forum on Centrifugal Pump Problems. Fairbanks, Morse & Co., Kansas City, Kans. (1954).

Chemical and Physical Characteristics of Delaware River Water from Trenton, N. J., to Marcus Hook, Pa., 1949–1952. BY C. N. DURFOR AND W. B. KEIGHTON.

Water Supply Paper 1262, U. S. Geological Survey. Price, $0.70 from Supt. of Doc., Govt. Printing Office, Washington, D. C. (1955).

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Proceedings of Member Associations

NEW ENGLAND SEWAGE AND INDUSTRIAL WASTES ASSOCIATION–NEW YORK SEWAGE AND INDUSTRIAL WASTES ASSOCIATION

The Fifth Joint Meeting of the New England Sewage and Industrial Wastes Association and the New York Sewage and Industrial Wastes Association was held at the Sheraton-Ten Eyck Hotel, Albany, New York, June 13–14, 1955. Registration totaled 230 members and guests, with 36 ladies also registered.

The technical program included the following papers:

"The Importance of Proper Piping in the Control of Sludge Digestion," by Edward S. Ordway, Engineer, Havens and Emerson, New York, N. Y.

"An Investigation of the Treatment of Cabin Cruiser Wastes," by William T. Ingram, Adjunct Professor, New York University.

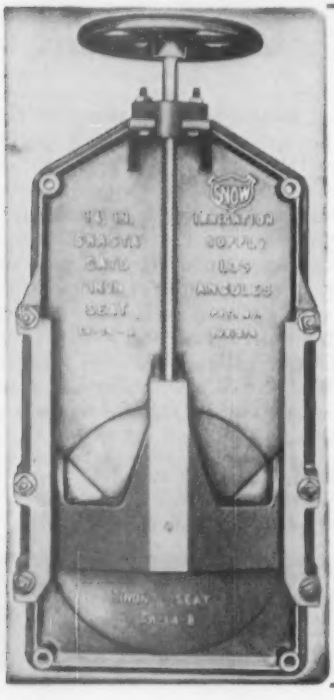
"Deodorizing Waste Water at the Avon Products Plant," by O. Davenport and B. Woods, Avon Products, Inc., Suffern, N. Y.

"The Treatment of Acid Wastes at the Worcester General Electric Plant," by V. deP. Lukas, Sanitary Engineer, General Electric Co., Schenectady, N. Y.

"The Fundamental Principles of the Control and Treatment of Wastes from the Dairy Industry," by H. A. Trebler, Consulting Chemical Engineer, Baltimore, Md.

"The Heat Disposal of Sludge," by Robert D. Nickerson, Eastern Sales

(*Continued on page 476a*)



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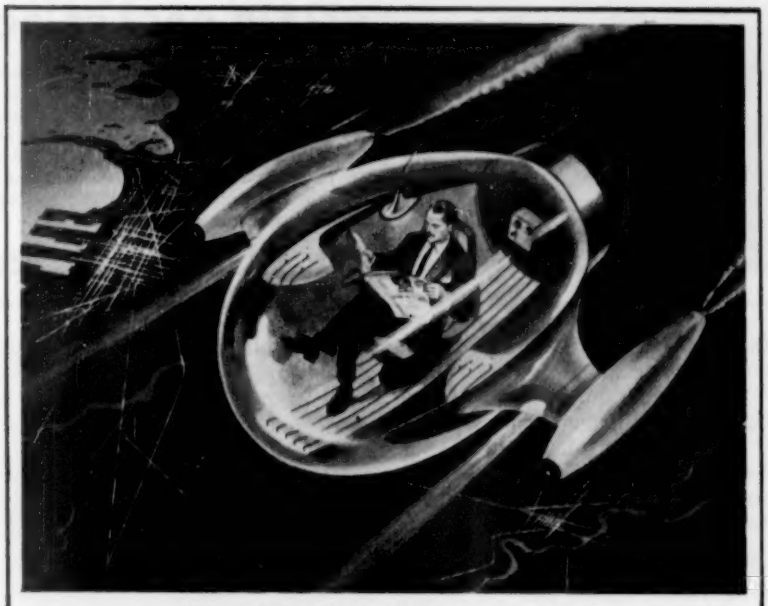

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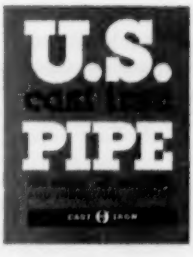

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Manager, Combustion Engineering, Inc., New York, N. Y.

"The Laboratory Evaluation of High Rate Sludge Digestion," by C. M. Sawyer, Professor of Sanitary Chemistry, and H. K. Roy, Graduate Student, Massachusetts Institute of Technology, Cambridge, Mass.

The second morning was devoted to an Operators' Round Table on the subject "Are Sewage and Waste Treatment Plants Today Being Designed on a Completely Realistic Basis?" L. H. Larkin, Chief, Water Pollution Control Section, New York State Department of Health, moderated the panel representing various aspects of the question.

Social activities featured a social hour and a luncheon. Guest speaker at the luncheon was Morris M. Cohn, Editor, *Wastes Engineering*, who spoke on "The City of the Future: A Challenge."

R. C. Sweeney,
Secretary, NYSIWA
Henry F. Munroe,
Secretary-Treasurer, NESIWA

MICHIGAN SEWAGE AND INDUSTRIAL WASTES ASSOCIATION

The 30th Annual Conference of the Michigan Sewage and Industrial Wastes Association was held at the Charlevoix Beach Hotel, Charlevoix, Mich., on June 20–22, 1955. Registration totaled 206, including 52 ladies.

The opening technical session featured a panel of 10 sewage and industrial waste plant operators who described some interesting phase of operation, construction or maintenance of their waste treatment plant.

Technical papers presented were:

"Employee Education," by C. T. Gruner, Supervisor of Plant Engineering, Detroit Diesel Engine, General Motors Corp., Detroit.

"Considerations in Handling Sludge From Industrial Waste Treatment Processes," by George A. McBride, Manager, Detroit Office, Infilco, Inc.

"A Synthetic Media for Trickling Filters," by Stanley J. Mogelnicki, Engineer, Dow Chemical Co., Midland.

In concurrent sessions the Industrial Waste Group heard a panel discuss "If I Were to Build Another Waste Treatment Plant" while the Sewage Treatment Group featured five small round table discussion groups to consider common problems of operation and maintenance of specific sewage treatment units.

The afternoon session on June 21 consisted of the annual business meet-

(*Continued on page 478a*)

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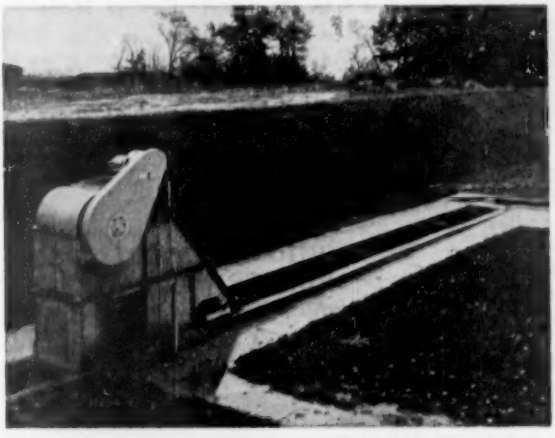

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ing, followed by an inspection trip to the Charlevoix Sewage Treatment Plant. Social events included the Annual Smoker, a Smorgasbord and dance, and the Annual Banquet followed by dancing.

Concluding sessions on June 22 featured an "Operators' Breakfast" and a series of general discussions by the entire group, moderated by J. C. Cooper, DuBois-Cooper Associates, Detroit.

Officers were elected as follows:

President: Milton P. Adams, Lansing.
1st Vice-President: Fred W. Galarno, Saginaw.
2nd Vice-President: George McBride, Gross Pointe.
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OHIO SEWAGE AND INDUSTRIAL WASTES TREATMENT CONFERENCE

The 29th Annual Meeting of the Ohio Sewage and Industrial Wastes Conference was held at the Hotel Deshler Hilton, Columbus, Ohio on June 22–24, 1955. Registered attendance totaled 226, including 22 ladies.

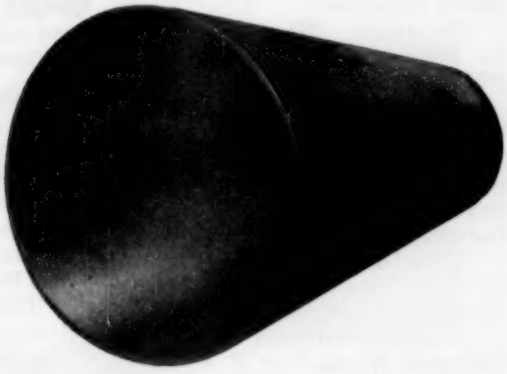
A Manufacturers' Forum was held the morning of June 22 preceding the official opening of the Conference. The forum featured "pump applications" with Paul M. Thayer, Chicago Pump Co., discussing sludge and grit; Spaulding A. Norris, Yeomans Brothers Co., discussing special applications; and Allen Mackey, Fairbanks, Morse and Co., discussing liquid waste applications. Moderator was Franklin Ruck.

Papers presented at the general sessions of the meeting included:

"Water Pollution Control in Ohio,"

(*Continued on page 480a*)

Gain 4 Ways with New Armco SMOOTH-FLO Sewer Pipe



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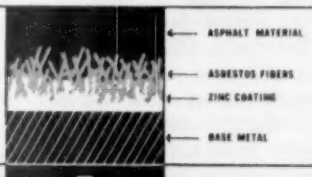
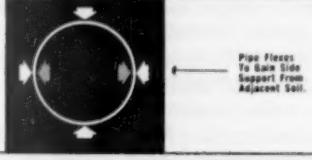


2. Proved Material Durability

The special-grade bituminous lining withstands heat or cold without damage. The base metal itself has extra protection from corrosion. Millions of asbestos fibers are embedded in the zinc coating and sealed with a special saturant. The result is Armco ASBESTOS-BONDED, a material proved in the most severe sewer service.



3. Corrugated Metal Strength

Armco SMOOTH-FLO Sewer Pipe has the flexible strength of corrugated metal design to eliminate the danger of cracking, breaking or crushing.

Coupling Band Formed To Fit Corrugations.

4. Strong, Tight Joints

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by Art Robinson, Director of Public Information, Water Pollution Control Board, Columbus.

"Federation Affairs," by Ralph E. Fuhrman, Executive Secretary, FSIWA.

"Modern Instrumentation Application and Control of Sewage Treatment Processes," by F. L. Mascitti, Instrumentation Engineer, Water Works and Sewage Dept., Minneapolis-Honeywell Regulator Co.

"Columbus Improvements," by J. H. Blodgett, Superintendent, Sewage Treatment Plant, Columbus.

"Automatic Sampling of a Variable Industrial Wastes," by W. G. Pfitzer, Procter and Gamble Laboratories.

"Highlights of the 1955 Ohio Water Clinic Conference," by Kenneth W. Cosens, Professor of Civil Engineering, Ohio State University.

Concurrent sessions were held by the sewage treatment and industrial wastes sections on June 23. Featured in the Sewage Treatment Section was an Operators' Forum, discussing select subjects from a recent operators' survey. In addition, the following papers were presented:

"Operation and Control Procedures," by J. D. Wittenmyer, Superintendent, Water and Sewage, Montgomery County.

"How to Avoid Sour Digester Problems," by D. P. Backmeyer, Superintendent, Little Miami Sewage Works, Cincinnati.

"A Description and Some Characteristics of Detergents," by Dr. George Malaney, Dept. of Bacteriology, Ohio State University.

"Improvements in the Activated Sludge Process," by L. S. Kraus, Chemist, Greater Peoria Sanitary District, Peoria, Ill.

"Further Study on Accelerated Sludge Digestion," by Ray Klingbeil, Assistant Superintendent, Sewage Treatment Plant, Columbus.

The Industrial Wastes Section opened with a forum discussion on the consulting engineer and industry. Featured was a panel of consulting and industrial engineers.

Papers presented were:

"The Water Pollution Abatement and Control Program of the Mahoning River Basin," by Bruce M. McDill, Engineer-in-Charge, and John E. Richards, Senior Assistant Engineer, Sewage and Industrial Wastes Unit, Division of Sanitary Engineering, Ohio Department of Health, Columbus.

"Chromic Acid Purification and Recovery by Ion Exchange," by L. M. Corcoran, Facilities Engineer, General Electric Co., Cincinnati.

"Miami River Basin Problems," by C. J. Velz, Chairman, Dept. of Public Health Statistics, University of Michigan.

"Oil Removal by Flotation," by R. N. Simonsen, Technical Service Division, Standard Oil Co., Cleveland.

Inspection trips were made to the Columbus Sewage Treatment Plant and to the Ranco, Inc. waste treatment plant, Delaware, Ohio.

Social activities included a cocktail party and the Annual Dinner and Dance. Guest speaker was R. J. Anderson, Assistant to the Director, Battelle Memorial Institute, Columbus.

The Bedell Award was presented in absentia to T. C. Schaetzle. The Dean Stewart Award and the J. W. Ellms Award were presented for the first time to Ben Barton, Findlay, and G. E. Flower, Cleveland, respectively.

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Pulp Flotation Machine—An air flotation unit to handle large volumes of pulp is featured in Bulletin F10–B87.—Denver Equipment Co., Box 5268, Denver 17, Colo.

Pneumatic Mechanisms Handbook—A new handbook-type bulletin on pneumatic control and transmission systems describes their application and operation. In addition, Bulletin 1120 contains practical operational information on instrument adjusting and tuning.—Minneapolis-Honeywell Regulator Co., Wayne and Windrim Aves., Philadelphia 44, Pa.

Programmer Thermometers—Specifications Sheet 602 describes internal cam programmer thermometers.—Minneapolis-Honeywell Regulator Co., Wayne and Windrim Aves., Philadelphia 44, Pa.

Automatic Ratio Relay—Bulletin 8410 describes an automatic ratio relay for pneumatic signaling in ratio and cascade controls.—Minneapolis-Honeywell Regulator Co., Wayne and Windrim Aves., Philadelphia 44, Pa.

Wrought Iron Pipe Catalog—New 52-page manual contains technical information, data, and tables on the characteristics and properties of wrought iron pipe.—A. M. Byers Co., Pittsburgh, Pa.

Rotary Pump—A positive displacement rotary pump with replaceable rubber impellers is described as self-priming and versatile. Harsh abrasives reportedly pose no problem.—Lindquist Pump Corp., 2207 Border Ave., Torrance, Calif.

Thermocouple and Pyrometer Supplies—New 56-page Bulletin P1238 on thermocouples and pyrometer accessories has just been published as a buyer's and user's guide.—The Bristol Co., Waterbury 20, Conn.

Anti-Rust Paint—A black anti-rust paint CI–88, incorporating a bituminous base, may be used as a finish coating and dries to a semi-gloss finish.—Chem Industrial Co., 3784 Ridge Rd., Brooklyn 9, Ohio.

Wedge Valve—A new wedge valve incorporates the sealing action of a line blind with the simplicity of gate valve operation, while at the same time providing full visibility of the wedge.—Hamer Valves, Inc., 2919 Gardenia Ave., Long Beach, Calif.

Phenolic Coatings—Bulletin 600 describes applications for modified phenolic coatings for use in tank linings, maintenance painting, and floor coverings in acid, alkali, and solvent environments.—Carboline Co., 331 Thornton Ave., St. Louis 19, Mo.

Tanks—Construction features, dimensions, and applications of wood, and welded or bolted steel tanks are illustrated in the new 16-page Bulletin T2–B5.—Denver Equipment Co., P. O. Box 5268, Denver 17, Colo.

Chemical Feeder—The low-capacity chemical feeder, "Chlor-O-Mite," for system flows less than 25 g.p.m. is described in Bulletin 1910–3.—Proportioneers, Inc., B–I–F Industries, Providence, R. I.

Universal Recorder—Bulletin FS–251 gives facts and figures on a new laboratory instrument that provides automatic recording for manual laboratory instruments.—Fisher Scientific Co., 711 Forbes St., Pittsburgh 19, Pa.

Asbestos Sheeting—Structurally strong sheet material of asbestos fiber for ready use in ovens, driers, breechings and housings is described in a 16-page, illustrated booklet.—Johns-Manville, 22 East 40th St., New York 16, N. Y.

Electric Motorized Valves—Bulletin 8506 presents specification data for the selection of valve bodies, linkages,

and power units for electric motorized valve assemblies.—Minneapolis-Honeywell Regulator Co., Wayne and Windrim Aves., Philadelphia 44, Pa.

Portable Diaphragm Pumps—Wheel mounted portable diaphragm pumps with single or double units are described in Bulletin 55DP.—Rice Pump and Machine Co., Belgium, Wis.

Explosion-Proof Motors—New explosion-proof electric motors, designed to meet Underwriters' specifications, are described in Bulletin No. 1879.—U. S. Electrical Motors Inc., Box 2058, Los Angeles 54, Calif.

Vertical Electric Motors—Bulletin No. 1868 describes improved types of vertical solid shaft electric motors.—U. S. Electrical Motors Inc., Box 2058, Los Angeles 54, Calif.

Chemical Feed Pumps—Reciprocating pumps for precise feeding of chemical solutions are described in Catalog S–1254.—Philadelphia Pump and Machinery Co., Wynnewood, Pa.

Chemical Pumps—Seal-less, leakproof centrifugal pumps for chemical service are described in Bulletin 1010.—Chempump Corp., Station E, 1300 E. Mermaid Lane, Philadelphia 18, Pa.

Rotary Vacuum Filters—Performance characteristics and technical details of continuous rotary vacuum filters featuring string discharge are given in the 24-page Bulletin No. 103–B.—Filtration Engineers, Inc., 155 Oraton St., Newark 4, N. J.

Mechanical Seals—Bulletin CP551 describes "wedge-lock" mechanical seals designed for interchangeability with packing in pump stuffing boxes. Successfully used on pumps handling acids, caustics, solvents, and slurries.—Chemical and Power Products, Inc., 11 Broadway, New York 4, N. Y.

Combination Clarifier-Digester—Illustrated Bulletin 135 describes the uses and features of the "Spiragester," a combination clarifier and digester embodying the principles of spiral-flow clarification with the simplicity of the Imhoff tank.—Lakeside Engineering Corp., 222 West Adams St., Chicago, Ill.

Liquid Level Gauges—Bulletin L701 comprehensively covers complete line of liquid and water level gauges.—The Bristol Co., Waterbury 20, Conn.

Bronze Valves—Bronze valves and fittings can be quickly selected through the use of Catalog No. B255.—Milwaukee Valve Co., 2375 South Burrell St., Milwaukee 7, Wis.

Steel Conduit—A new booklet describes heavy-duty, rigid steel electrical conduit especially designed for industrial use requiring corrosion-resistant materials.—National Electric Products Corp., Gateway Center, Pittsburgh, Pa.

Thermostatic Controls—The condensed Bulletin C–6 covers thermostatic control valves and accessories.—Lawler Automatic Controls, Inc., 453 North MacQuesten Parkway, Mount Vernon, N. Y.

Filter Block—A redesigned, two-piece filter block having a considerably increased strength over ordinary block can be used to eliminate hand placement of filter stone.—Straitsville Brick Co., New Straitsville, Ohio.

Pipe Markers—Self-adhesive pipe markers insure permanent identification without being effected by heat, cold, humidity, or vibration.—North Shore Nameplate Inc., 214–27 Northern Boulevard, Bayside, Long Island, N. Y.

Instrumentation—Composite Catalog 5002 briefly and concisely describes principal types of industrial indicating, recording, and control instruments.—Minneapolis-Honeywell Regulator Co., Wayne and Windrim Ave., Philadelphia 44, Pa.

Pneumatic Controller—Bulletin 5A–13 details a new model pneumatic indicating controller incorporating a proportioning action adjustment.—The Foxboro Co., Foxboro, Mass.

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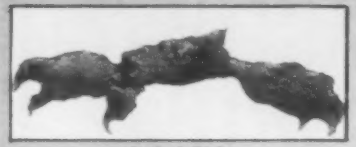

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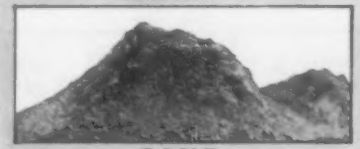

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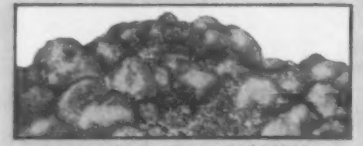

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